Chapter Nine

Rehabilitation of Existing Bridges

9.1 INTRODUCTION

The Department evaluates several factors in the identification and prioritization of bridge rehabilitation candidates. These include:

- bridge sufficiency rating,
- historical significance,
- traffic volumes/ADT,
- bridge load capacity posting,
- bridge condition, including emergency repairs,
- length of detour, and
- economic analysis considering
    - repair,
    - rehabilitation,
    - replacement, and
    - life-cycle costs.

Sources for the above information include:

- the Bridge Management System (Pontis),
- the bridge safety inspection file and structure data records (SDR),
- traffic counts from Division of Planning,
- Delaware's historic bridges survey and evaluation,
- bridge load ratings, and
- the Bridge Paint Priority Listing.

Bridges are selected for improvement in accordance with those rankings. Most bridge rehabilitation projects are initiated through these rankings.

All Delaware bridges in the National Bridge Inventory (NBI) (bridges with spans greater than 20 feet [6.1 m]) are inspected periodically (at least every other year) to evaluate their condition and ensure the safety of the public. Bridges not on the NBI are inspected at least once every five years. Refer to Section 2.1 for size requirements for non-NBI bridges. Bridges not owned by the state are inspected by their owners. Bridges must be inspected to be eligible for federal-aid funds. Information from the inspections is entered into the Bridge Management System (BMS). The Department uses the Pontis Bridge Management System, which was developed through an AASHTO/FHWA joint development effort. Pontis provides a comprehensive ranking based on benefit/cost ratios for each bridge. It includes preventative maintenance strategies as well as replacement and rehabilitation options. Pontis is also used for multi-year optimizations for use in projecting budget needs.

For each bridge, the Bridge Management Section performs an NBI inspection and a Pontis element level inspection. NBI inspections provide an overview of the major bridge areas, i.e., deck, superstructure, and substructure, as well an appraisal of the structural adequacy and functionality of the bridge. Pontis element level inspections provide detailed inspection reports of each element on the bridge. Each element is divided into various condition states, representing differing degrees of deterioration. This information is used by the Pontis program to compute repair costs and benefits. DelDOT's use of the Pontis program will only address maintenance, repair, rehabilitation, or replacement recommendations based on element condition; the Pontis program will not address functional deficiencies. The Bridge Management Section runs a Pontis scenario, which optimizes and prioritizes repair recommendations for all DelDOT owned bridges. A list of the bridges will be generated which will include element quantities, recommended work, and cost estimates. The list will be screened for work that can be performed by the Districts, either by in-house forces or by structure maintenance contracts. The remaining bridges will be sent to the Bridge Design Section for investigation. This investigation will include a review of the inspection reports and a field review. Based on this list, bridges are added to the Bridge Design program, and are initiated as projects. Most Bridge Design projects are initiated through the Bridge Preservation Program.

Some bridges may have to be rehabilitated because of unforeseen emergency circumstances, such as fire damage, washouts, or structural damage from traffic. The priority given these emergency projects will depend on their impact on traffic, the ease of detouring traffic, and the severity of the deficiency. Eligibility for federal funding will be determined on a case-by-case basis. Refer to Section 2.3.3.

The Department relied heavily on the John Wiley & Sons, Inc. publication *Bridge Inspection and Rehabilitation: A Practical Guide* by Parsons Brinckerhoff, for remedial methods described in this chapter. This information is used with the permission of John Wiley & Sons, Inc.

9.2 ENVIRONMENTAL CONSIDERATIONS

The Department encourages recycling materials obtained from demolition of structures and roadways. However, the designer must be constantly aware of the environmental aspects of bridge rehabilitation. Many older structures utilized materials in their construction that are environmentally unacceptable today.

The designer must be aware of environmental permit requirements and conditions for removal and disposal of such materials or their by-products. Refer to Section 2.9 for regulatory agencies involved in the permit and approval process.

9.2.1 REPAINTING

The designer should check the reports from the Bridge Management System to determine whether the bridge steel is coated with lead-based paint. A request should be made to Materials and Research for testing of the existing paint to verify that it is lead based.

Before structural steel can be repainted, the steel must be cleaned. Cleaning steel to remove existing coatings involves airborne blast dust and blast by-product collection and disposal. The Department requires 100 percent containment of blast by-products during cleaning according to the contract specifications. The coatings contractor is required to submit a containment plan and recovery system for approval prior to initiating work. Residue from non-hazardous-based paints is always classified as industrial waste. Residue containing hazardous-based paint is considered a hazardous material and must be tested. Handling and disposal of either hazardous material or industrial waste must be in accordance with State Department of Natural Resources and Environmental Control (DNREC) regulations. Wash water from cleaning structural steel must be handled and disposed of in accordance with contract specifications.

Another environmental concern is control of painting fumes. The Department specifies paints with low levels of volatile organic compounds (VOC) to reduce fumes.

9.2.2 RECYCLING

Recycling materials helps to conserve natural resources. For recycling hot-mix, see the Road Design Manual.

In an effort to recycle concrete construction materials, the designer is encouraged to utilize removed concrete for base materials for approach roadways or as riprap for slope protection, where possible. Broken concrete must meet the specification requirements for riprap.

9.2.3 DEMOLITION

The designer must evaluate alternate demolition methods, including the need to provide demolition shields, during design and specification development to ensure that the demolition can be performed using normal methods both safely and economically. Demolition methods are the responsibility of the contractor. The Department reviews demolition methods and shields in the course of pre-construction reviews and during construction.

9.3 BRIDGE DECKS

Most bridges that require rehabilitation have concrete decks. However, some bridges have steel grid decks or timber decks.

9.3.1 CONCRETE DECKS

9.3.1.1 Condition Survey

The designer has a wide range of destructive and non-destructive tests available for determining the condition of an existing concrete bridge deck. These include:

- visual inspection,
- chloride content analysis,
- freeze-thaw resistance test,
- half-cell analysis,
- delaminations survey,
- coring, and
- petrographic analysis.

The designer must request any needed tests (except visual inspection) from the Materials and Research Section. Determination of the proper deck rehabilitation strategy requires an evaluation of all test results and should not be based on the results of any one test.

9.3.1.1.1 Visual Inspection

Visual inspection of the condition of a concrete deck involves the assessment of five major factors:

- **Spalling** is caused by the separation and removal of a portion of the concrete leaving a roughly circular or oval depression in the concrete.

- **Scaling** is the gradual and continuous loss of surface mortar and aggregate.

- **Cracking** is a linear fracture of the concrete and may extend partially or completely through the concrete member.

- **Efflorescence** is a chemical reaction that results in the deposition of a powdery crystalline substance that appears on the surface of the concrete. It is caused by the evaporation or chemical change of the concrete. Efflorescence is an indication of a localized high chloride concentration and leakage.

- **Alkali silica reactivity** (ASR) is the susceptibility of certain aggregates to chemically react with the alkalies, such as sodium and potassium, in Portland cement concrete. A reactive silica material is one that reacts at high temperatures with Portland cement or lime. This type of material includes pulverized silica, natural pozzolan, and fly ash. Under certain conditions, harmful expansion, cracking, and staining of the concrete can occur.

- **Delayed ettringite formation in concrete**

Each factor should be evaluated to determine the percent of deck area in each span that exhibits these conditions. Finally, an overall percentage for each condition should be computed for the total bridge deck.

Both the top surface and the underside of the concrete deck should be visually inspected. Often the presence of a bituminous concrete wearing surface will prevent the visual inspection of the top of the deck. In these cases, the wearing surface may be partially or totally removed and the deck inspected. The visual inspection may also be based on the deck's underside condition only.

9.3.1.1.2 Chloride Content Analysis

The amount of soluble chlorides in a concrete deck is a quantitative measure of the potential of the concrete to react electrochemically with the embedded reinforcing steel.

To determine chloride content, the concrete deck is drilled, and samples of the concrete powder are collected and analyzed from various depths of the deck (1 in [25 mm], 2 in [50 mm], 5 in [125 mm], and at the depth of the top steel reinforcement if it is deeper than 5 in [125 mm]) and analyzed for chloride concentration. Sampling and testing should comply with AASHTO T260-78. The results of these tests are used in conjunction with half-cell analyses to define potential deck rehabilitation options. Rehabilitation options may apply to small areas of the deck or the entire deck.

Generally, the criteria in Figure 9-1 are used to assess the deck for chloride concentration levels.

Figure 9-1
Chloride Concentration Criteria

Chloride Concentration	Level of Chloride Contamination
0 to 1.3 lb/cu yd (0 to 0.8 kg/m^3)	Low
1.3 to 2.0 lb/cu yd (0.8 to 1.2 kg/m^3)	Moderate
2.0 lb/cu yd (1.2 kg/m^3) or greater	Advanced

Usually, chloride concentration levels in the top 2 inches [50 mm] of deck thickness will be higher than those in other parts of the deck. Chloride concentration is not, by itself, an indication of corrosion activity or unacceptable concrete strength. Remedial corrective action will be different depending on the chloride ion concentration at each level in the deck. Also, chloride concentration levels will typically be higher in the gutter area than in other areas of the deck. Consequently, different areas of the deck may require different actions. Chloride content should not be the sole determining factor for deck replacement.

9.3.1.1.3 Freeze-Thaw Test

The vulnerability of concrete to damage from freeze-thaw cycles is a measure of concrete durability. The AASHTO T-161 test method measures the resistance of concrete to rapid freezing and thawing and indicates the ability of the concrete to withstand freeze-thaw cycles. This test method covers determination of the resistance of concrete specimens to rapidly repeated cycles of freezing and thawing in the laboratory by two different procedures: rapid freezing and thawing in water, and rapid freezing in air and thawing in water. Both procedures are intended for determining the effects of variations in the properties of concrete on the resistance of the concrete to the freeze-and-thaw cycles. Neither procedure is intended to provide a quantitative measure of the length of service that may be expected from a specific type of concrete.

The first step in the procedure is to determine the initial dynamic modulus of elasticity for the concrete specimen. The specimen is then subjected to freeze-thaw cycles. Additional dynamic modulus of elasticity tests are taken periodically during the test. The test continues until its dynamic modulus of elasticity reaches 60 percent of the initial dynamic modulus or a maximum of 300 cycles. The final dynamic modulus of elasticity is used to determine the Durability Factor (expressed as a percentage) for the concrete specimen.

A Durability Factor with a value of 60 percent or lower indicates that the concrete is no longer in an acceptable condition for its intended purpose. This implies that some repairs to the deck would be required, although not necessarily to the extent of redecking.

9.3.1.1.4 Half-Cell Analysis

A half-cell analysis measures the active corrosion and corrosion potential of embedded reinforcing steel. This is done by measuring the electrical potential between two points. Half-cell readings are usually taken on the concrete deck surface along a grid pattern at 4 foot [1.2 m] intervals. As part of the half-cell analysis test procedure, it is necessary to attach one wire of a high-impedance voltmeter to the deck reinforcing steel. The second wire is attached to a copper-sulfate half cell. This test procedure should comply with ASTM C-876.

A pachometer is used to locate the reinforcing steel for the half-cell analysis. The pachometer identifies the location and depth of the reinforcing steel using the magnetic field disruption principle. The pachometer can also be used to determine the depth of cover over the deck reinforcing steel throughout the deck. This is valuable in evaluating deck service life and rate of deck deterioration. (Decks with bituminous concrete wearing surfaces cannot be tested unless the concrete is exposed.)

The measurement of the flow of electrons is used to evaluate corrosion activity. See Figure 9-2.

Figure 9-2
Corrosion Evaluation Criteria

Half-Cell Reading (Volts)	**Level of Corrosion**
0.0 to -0.35 V	Insignificant or no corrosion
-0.36 to -0.50 V	Minor corrosion
Greater than -0.50 V in magnitude	Major corrosion

The results from the half-cell analysis are used in conjunction with other test results to evaluate the overall condition of the deck.

9.3.1.1.5 Delamination Survey

Delamination is the separation between "layers" of a concrete deck in the horizontal plane. Testing for delaminations can be performed by sounding with a hammer or using a chain drag. The chain drag relies on an inspector's ability to recognize hollow-sounding areas of the deck when the device is dragged along the top of the deck. Sounding with a hammer can further delineate the hollow areas.

Recent advances in technology have resulted in other methods to detect and record deck delaminations. These include:

- **Ground-penetrating radar** utilizes radar technology to investigate the bridge deck concrete. It provides a profile of subsurface layers by evaluating signal reflections caused by discontinuities. This method can be used for delamination testing on a full lane width in a single pass at up to 30 mph [50 km/h].
- **Infrared thermography** detects locations of delaminations by measuring the temperature differences between solid concrete and delaminations (air).
- **Micro-seismic** measures the intensity of an echo from a small impact as it is transmitted through the concrete and reflected by discontinuities.

Materials and Research does not have the equipment to perform ground-penetrating radar, infrared thermography or micro seismic evaluations of concrete. However, should these evaluations be needed, these services can be provided by consultants.

9.3.1.1.6 Coring

Coring bridge decks can allow the designer to evaluate the compressive strength of the sample. Cores are also visually inspected to determine the erodeability and "weathering" of the cement matrix and aggregate as a result of the coring process. AASHTO Test Procedure T-24 should be followed for all coring.

The designer should review all the compressive strength test results to identify the high- and low-strength areas and the concrete variability throughout the deck. It may not be necessary to remove the concrete in the high strength areas. Removal of higher strength concrete is more difficult, and the designer must alert the contractor to the variability of the concrete strength, especially when hydro-demolition is used.

9.3.1.1.7 Petrographic Analysis

Petrographic analysis is a microscopic examination of a concrete sample. It identifies:

- voids,
- micro-cracking in coarse aggregate, and
- cracking or debonding between the aggregate and the cement grout,

and can be used to determine:

- unit weight,
- moisture content, and
- specific gravity.

9.3.1.2 Deck Condition Evaluation

The deck condition survey data must be evaluated to determine the suitability of the deck for repair, rehabilitation, or replacement.

The FHWA has established three standard categories for concrete deck deterioration where the chloride content tested greater than 2 pounds water-soluble chloride per cubic yard concrete [1.2 kg water-soluble chloride per cubic meter concrete] for samples taken above the top steel, and 1 pound chloride per cubic yard concrete [0.6 kg chloride per cubic meter concrete] for samples taken between the top and bottom mats of steel. These categories are defined below and summarized in Figure 9-3.

Category 1: Extensive Active Corrosion. Minimum of 5% of the deck visibly spalled or minimum of 40% of the deck area having deteriorated or contaminated concrete, or active rebar corrosion, as indicated by a summation of nonduplicating areas consisting of spalls, delaminations, electrical potentials with negative readings in excess of -0.35 V. Category 1 decks are normally replaced.

Category 2: Moderate Active Corrosion. Zero to 5% of the deck visibly spalled, or 5 to 40% of the deck area having deteriorated and/or contaminated concrete, and/or active rebar corrosion, as indicated by a summation of nonduplicating areas consisting of spalls, delaminations, electrical potentials with negative readings in excess of -0.35 V. These decks should be rehabilitated.

Category 3: Light or No Active Corrosion. No visible spalls, or 0 to 5% of the deck area having deteriorated and/or contaminated concrete, and/or active rebar corrosion, as indicated by a summation of nonduplicating areas consisting of spalls, delaminations, electrical potentials with negative readings in excess of -0.35 V. These decks should be rehabilitated.

Figure 9-3
Bridge Deck Evaluation Criteria

<table>
<tr><th rowspan="3">Category Classifications</th><th colspan="4">Condition Indicators (Percent of Deck Area)</th></tr>
<tr><th colspan="2">Deterioration</th><th colspan="2">Contamination</th></tr>
<tr><th>Spalls</th><th>Delaminations</th><th>Electrical Potential</th><th>Chloride Content</th></tr>
<tr><td>Category 3 Light Deterioration</td><td>None</td><td>< 5</td><td>5% < -0.35 V</td><td rowspan="4">> 2.0 lb/cu yd (1.2 kg/m³) above top steel and > 1.0 lb/cu yd (0.6 kg/m³) between steel mats</td></tr>
<tr><td>Category 2 Moderate Deterioration</td><td>< 5</td><td>5-40%</td><td>5-40% > -0.35 V</td></tr>
<tr><td rowspan="2">Category 1 Extensive Deterioration</td><td colspan="3">The sum of all deteriorated and/or contaminated deck concrete is less than 40%</td></tr>
<tr><td>> 5</td><td colspan="2">The sum of all deteriorated and/or contaminated deck concrete is greater than 40%</td></tr>
</table>

Decks with low chloride contamination should be waterproofed using silene sealer as a minimum treatment. Decks with moderate chloride contamination may be overlaid to extend their service life. Decks with advanced chloride contamination should be considered for replacement only if the condition is poor.

The above categories are guidelines. Traffic impacts, costs of temporary overlay alternates, and service life expectancy (rate of deck deterioration) should also be evaluated.

9.3.1.3 Concrete Removal

Three methods are used for removal of concrete for rehabilitation of decks:

- jackhammer,
- hydrodemolition, and
- milling.

The edges of areas to be patched must be saw cut 1 inch [25 mm] deep into squares or rectangles. Saw cuts must be stopped at the corners to prevent overcutting. The corners must be hand chipped. The rest of the removal is performed with jackhammers, hydrodemolition, or hand chipping.

Jackhammer. The size of the jackhammer must be appropriate for the amount of removal to prevent unnecessary damage to the deck.

Hydrodemolition. Hydrodemolition is the use of high-pressure water jetting on a large scale to remove deteriorated concrete from bridge decks. The extent of concrete removal is primarily determined by concrete strength, water pressure, type of nozzle, and equipment speed. The designer must consider the strength of the concrete and the capability of the equipment before specifying this method of removal. Sufficient deck condition data must be obtained to evaluate the removal needs. The designer must specify the minimum depth of concrete removal in areas with high concrete strengths and estimate the quantity of removal in all areas.

Excessive pressure or inappropriate machine speeds will result in the removal of an excessive depth of concrete. To prevent this, the contractor is required to perform a demonstration in a test section. The designer must determine the size and number of test sections. Multiple machine settings may be required to match the depth of removal with the levels of deterioration and concrete strengths.

The depths and limits of removal and the number of test sections must be shown on the plans.

Milling. Milling is used to prepare decks for complete overlays. The weight of the milling machine must be considered when milling bridges constructed with low or highly variable deck concrete strengths. The depth of rebar cover should be determined using a pachometer in order to avoid damage to the rebar by the milling machine.

9.3.1.4 Concrete Deck Rehabilitation

Concrete deck rehabilitation involves patching, overlay or lithium treatment for ASR. An economic analysis considering partial- and full-depth patching, partial-span deck replacement, overlay and a combination of these actions should be performed before a final action is defined.

9.3.1.4.1 Patching

Bridge decks in Category 2 and 3 (moderate active corrosion and light or no active corrosion), Section 9.3.1.2, are candidates for patching and overlaying.

Frequently, a bridge deck does not require complete replacement. If the deck condition survey data indicates that the majority of the deck is in good condition with only the top surface requiring replacement (due to spalling or high chloride concentration) and with other areas of the deck requiring only partial or limited full-depth patching, the deck can be rehabilitated by patching and overlaying. In this case, the top surface of the deck, above the top mat of reinforcing steel, is removed without debonding, damaging or dislodging the reinforcing steel. Removal can be accomplished by scarifying, hand chipping, or hydrodemolition. All deteriorated and high chloride contaminated concrete must be removed.

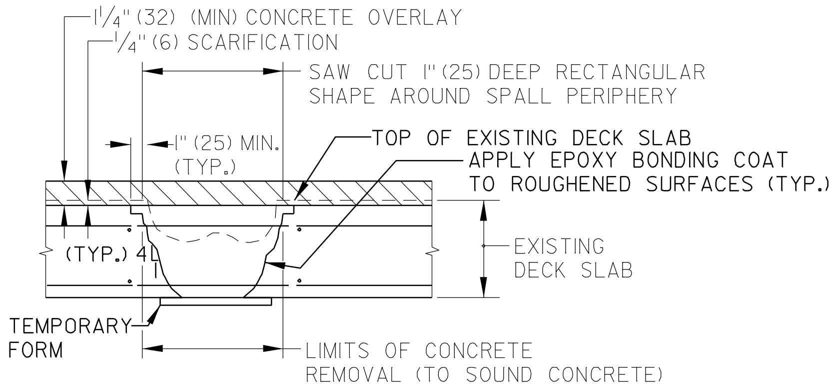
After the surface concrete is removed, all remaining areas of highly deteriorated concrete are removed (hand chipped) and replaced. Patching material must be compatible with the selected overlay. The surface must be patched first so the overlay will be of uniform thickness. Finally, an overlay is applied to the full deck surface and may increase deck thickness. A structural analysis is necessary before an overlay is installed because of the increased dead load.

Any of the overlays listed in Section 9.3.1.4.2, except bituminous overlays, may be used to repair decks after the patching is completed.

9.3.1.4.1.1 Repair by Patching

Bridge decks in Category 3 (light or no active corrosion), Section 9.3.1.2, are candidates for repair. The method of deck repair depends on the depth of the deteriorated area. A shallow repair is used where the depth of concrete deterioration is less than 2 inches [50 mm] and reinforcing is not exposed. Deep repairs involve removal of concrete below the reinforcing steel, cleaning of the steel, supplementing deteriorated steel, and placing and curing the repair material. Generally, deep, permanent concrete repair material, such as Class D concrete, is used. Insofar as it is practical, the concrete repair material should have the same strength, permeability, and expansion/contraction characteristics as the existing concrete. Figure 9-4 illustrates a full-depth deck repair.

Figure 9-4
Full Depth Deck Repair



9.3.1.4.2 Overlay

Bridge decks in Category 2 and 3 (moderate active corrosion and light or no active corrosion), Section 9.3.1.2, are candidates for overlaying.

Five types of overlays are used for deck rehabilitation:

- latex-modified concrete,
- silica fume concrete,
- other thin-bonded concrete overlays,
- Class D concrete, and
- bituminous concrete.

Designers should compare all methods and materials to select the optimum one for the particular application. The minimum total cover over the top mat of reinforcing steel is 2.5 inches [65 mm] for any type of cementitious overlay. Surface preparation is necessary to ensure the required bond between the overlay and the deck.

A structural analysis is needed before applying any overlay that increases the dead load.

Latex-modified concrete (LMC) consists of cement mortar or concrete mixed with styrene-butadiene latex. LMC may be used for thin patches, which may be made monolithically with the overlay. The minimum thickness of an LMC overlay is 1.25 inches [32 mm].

Silica fume concrete consists primarily of noncrystalline pozzolanic silica. It has many of the same characteristics as latex-modified concrete but is more economical and has a longer curing time.

Other thin-bonded concrete overlays include polymer concrete and other proprietary products.

The maximum thickness for latex-modified concrete, silica fume concrete, and other thin-bonded concrete overlays is 2 inches [50 mm], but that may be increased if reinforcing is used to minimize cracking. Latex-modified concrete and silica fume concrete are normally bid as alternates in order to get the lowest possible overlay cost.

Class D concrete overlays are made with the same class of Portland cement concrete used for other superstructure elements. The minimum thickness of Class D concrete overlays is 3 inches [75 mm]. Where thicker overlays are needed, Class D concrete should be used.

Bituminous concrete overlays are not used by the Department for permanent repairs. They may be used for temporary repairs and to improve rideability. Bituminous overlays allow chlorides to penetrate to the concrete surface, hold moisture, and prevent washing the chlorides from the surface of the concrete. A membrane should be used if a bituminous concrete overlay is considered to be a permanent repair.

9.3.1.4.3 Waterproofing

Bridge decks and sidewalks may be treated with a water-repellent solution consisting of isobutyl-trimethoxy silane-based material. Because silane cannot be applied over curing compound, specifications must provide for water curing where this waterproofing is to be used. The need for waterproofing is determined on a case-by-case basis.

9.3.1.5 Concrete Deck Replacement

Bridge decks in Category 1 (extensive active corrosion; see Section 9.3.1.2), are candidates for replacement. Where concrete deterioration extends below the top mat of reinforcing steel over a large area of the deck, total deck replacement is necessary. Because the remaining superstructure and substructure must remain in service as long as the new deck, these elements must be in good condition. A final decision to proceed with a deck replacement depends on the condition and evaluation of remaining portions of the superstructure and substructure.

The most common deck replacement options include:

- cast-in-place concrete,
- precast concrete, and
- orthotropic deck.

Refer to Section 5.2 for design details for cast-in-place and precast decks.

9.3.1.5.1 Cast-in-Place Concrete

Cast-in-place concrete is the most common deck replacement option. Both normal weight and lightweight concrete have been used successfully. A lightweight concrete deck will decrease the dead load, thereby increasing the capacity of a structurally deficient bridge and allowing heavier live loads. The use of lightweight concrete must be based on the results of a structural analysis.

9.3.1.5.2 Precast Concrete

Decks may be replaced with precast deck slabs or deck panels. Precast deck slabs are cast full thickness in segments for placement on the beams. A concrete overlay must be placed to improve the ride. Precast deck panels are cast partial thickness and are placed to act as stay-in-place forms. The remainder of the deck is cast in place to form a full-thickness composite deck. Either precast slabs or deck panels may be conventionally reinforced or prestressed. Precast slabs or panels provide the following advantages:

- excellent quality control,
- flexibility of fabrication,
- fast construction, and
- reduced traffic delays and inconvenience to the traveling public.

They have the following disadvantages:

- potential for cracking at slab and panel joints, and
- design details more critical to fabrication and construction.

These methods require the designer to consider the following:

- composite action requirements,
- details for attachment of the slabs or panels to the superstructure,
- horizontal shear connection between the deck and superstructure,
- joints between slabs or panels,
- riding surface, and
- type of overlay.

Refer to Section 5.2.2 for design of precast slabs and deck panels.

9.3.1.5.3 Orthotropic Decks

An orthotropic deck can replace both the deck and the support stringers of a rehabilitated structure. The use of an orthotropic deck must be approved by the Bridge Design Engineer. An orthotropic deck consists of a stiffened steel deck supported by trusses or frames. The deck acts as the top flange. An orthotropic deck permits increasing the spacing of primary load-carrying members to a range of 15 to 40 feet [4.6 to 12.2 m]. The designer should evaluate the impact on profiles and clearances when considering an orthotropic deck on existing support elements. Refer to Section 9.8 in the *AASHTO Specifications* for design of orthotropic decks.

Wearing surfaces must be added to orthotropic decks to provide skid resistance and protect the steel. Epoxy asphalt wearing surfaces have performed better than bituminous concrete in recent applications. Deck plate flexibility has been a cause of wearing surface bonding failures.

Advantages of this system include improved quality control through shop fabrication, fast construction, and minimal traffic impact.

9.3.2 STEEL GRID DECKS

Steel grid decks are used only to replace existing steel grids, usually on movable bridges. Steel grids may be filled full depth or half depth with either normal weight or lightweight concrete. Overfilling the grid provides an integral wearing surface. A separate wearing surface may also be placed.

9.3.2.1 Steel Grid Deck Evaluation

There are five considerations in the evaluation of steel grid decks:

- failure of the connections of the grid to the superstructure,
- reduced skid resistance,
- fatigue evaluation,
- corrosion, and
- delamination.

Where these conditions become severe, replacement of the grid should be considered.

Steel grid deck rehabilitation can be accomplished by replacing all or portions of the grid.

9.3.2.1.1 Connection Failure

The connections of the various grid components are subjected to forces caused by the interaction between the grid and its supporting elements. These forces stem from vehicle loads, including those forces introduced through braking or accelerating. These connections will fail over time because of fatigue and other time-dependent effects. Failed connections can be identified through visual inspection. Refer to the bridge inspection file.

9.3.2.1.2 Reduced Skid Resistance

Both open grids and concrete-filled grids without surfacing are subject to decreased skid resistance over time. Unsurfaced filled grids can develop cupping or wear of the concrete between the grid bars, which exposes the grid to direct wheel loads. The surface of the filled grid then becomes similar to that of an open grid, and skid resistance quality declines. In wet weather, this is dangerous as water is held in the cups. In freezing weather the hazard increases due to ice formation. The riding surface of the grid elements, when new, presents some resistance to skidding, but wear causes a reduction in skid resistance.

9.3.2.1.3 Fatigue Evaluation

Fatigue is defined as the damage that may result after a sufficient number of fluctuations of stress have occurred. When steel grid decks are subject to many cycles of loading and unloading (from 20,000 to over 5,000,000), the metal may fatigue and develop cracks in regions of high and localized stress. Left unaddressed, fatigue cracking can ultimately lead to the complete failure of steel members.

To evaluate the severity of the fatigue, the engineer must consider:

- the number of load applications,
- the magnitude of the stress range (Fs_{max} to Fs_{min}), and
- the stress concentrations associated with particular details.

Fatigue and cracking do not develop in compression members. Variations of stress that occur within tension members, however, will always be a major consideration in the evaluation of existing steel grid decks.

9.3.2.1.4 Corrosion

The grid bars (and the supporting purlins and stringers in the case of open grids) are exposed to road chemicals, including deicing salts, which cause corrosion to develop in the steel grid deck system.

9.3.2.1.5 Delamination of Surfacing

Filled and surfaced grids can also be subject to delamination between the riding surface and the grid.

9.3.2.2 Steel Grid Deck Replacement

Refer to Section 9.8.2, in the *AASHTO Specifications* for design of steel grid decks.

Steel grid decks should always be galvanized.

Replacement grid decks should be shifted along the primary support member to preclude welding the grid at the same locations as the previous welds. Attachment of the grid to the superstructure is a critical detail and must be closely evaluated.

9.3.3 TIMBER DECKS

9.3.3.1 Timber Deck Survey and Evaluation

Wood decks nearing the end of their useful lives exhibit a number of signs indicating that there are problems:

- excessive deflection under load,
- loose connections as a result of shrinkage,
- deterioration, such as checking, cracking, or crushing, and
- wear of the timber deck evidenced by protruding nails.

When the four survey conditions investigated above become severe, replacement of the timber members is required.

9.3.3.2 Timber Deck Replacement

Total timber deck replacement is rarely needed. Generally individual planks are replaced in kind as required by maintenance.

When support beams or nailers become deteriorated or will no longer hold the fasteners, the beams/nailers/sleepers and the complete deck must be replaced. Only timber planks pressure treated in accordance with the specifications with a preservative. Consideration should be given to using longitudinal deck panels only if the substructure is in good condition. Refer to Chapter Eight, Timber Design, and Section 9.9, in the *AASHTO Specifications* for design of timber decks.

A bituminous concrete overlay is generally placed over timber decks for rideability and to meet skid resistance criteria.

9.3.4 DECK PROTECTIVE SYSTEMS

The Department specifies Class D concrete with low water-cement ratios and integral wearing surface over the top mat of reinforcement to ensure long-lasting decks. Additional deck protection includes:

- waterproofing,
- cathodic protection,
- epoxy-coated reinforcing steel, and
- overlays.

9.3.4.1 Waterproofing

Bridge decks and sidewalks may be treated with a water-repellant solution consisting of an isobutyl-trimethoxy silane-based material. Because silane cannot be applied over curing compound, specifications must provide for water curing where this type of waterproofing is to be used.

9.3.4.2 Cathodic Protection

Reinforcing steel in chloride-saturated concrete creates an electrolyte when exposed to water. The difference in electric potential between dissimilar materials causes an electrochemical reaction resulting in electric current flowing from one electrode (anode) to the other (cathode). This process can cause rust to form which rapidly reduces the cross section of the reinforcing steel. The most commonly used method to reverse this process is cathodic protection.

Cathodic protection is suitable for chloride-contaminated decks with sound concrete. Cathodic protection accelerates alkali silica reactivity (ASR). The deck should be checked for ASR before considering the use of cathodic protection. Cathodic protection can only be used with the approval of the Bridge Design Engineer.

9.3.4.3 Epoxy-Coated Reinforcing Steel

The Department routinely uses epoxy-coated reinforcing steel in all new and replacement decks, slabs and parapets. Epoxy-coated bars should not be used to supplement uncoated reinforcing steel.

9.3.4.4 Overlays

Refer to Section 9.3.1.4.1 and Section 9.3.1.4.2.

9.3.5 SAFETY CONSIDERATIONS

It is desirable to update as many safety items as possible according to current Department and AASHTO standards when a deck rehabilitation or replacement project is constructed. These items include:

- barrier rail,
- approach guardrails and attachments to the structure,
- curbs and/or sidewalks, and
- approach guardrail end treatments.

Structures are widened to match approach roadway geometry wherever practical. Consideration must be given to providing minimum bridge width, appropriate shoulder widths, and betterment of the functionality of the structure to remove the bridge from the functionally obsolete classification.

A functionally obsolete bridge is one in which the deck geometry, clearance, or approach roadway alignment no longer meets the usual criteria for the system of which it is an integral part. Numerically in the FHWA NBI Coding Guide, a functionally obsolete bridge shall have an appraisal rating of 3 or less for:

- deck geometry (item 68),
- underclearance (item 69), or
- approach roadway alignment (item 72),

or an appraisal rating of 3 for:

- structural condition - deck (item 58), or
- waterway adequacy (item 71).

Refer to Section 5.2.7 in this manual; Chapter Ten, Miscellaneous Design, in the DelDOT *Road Design Manual*; and Section 2, General Design and Location Features, in the *AASHTO Specifications*.

9.4 RAILINGS

Bridge railings may be upgraded to current standards as part of a bridge rehabilitation project, wherever economically feasible and within the scope of the project. Standard types of rails are shown in Section 5.2.7.1. Types of railings include:

- concrete - F-shape,
- concrete - vertical wall with handrail,
- concrete - stone faced,
- steel or aluminum, and
- timber.

Bridges with sidewalks or safety walks may be retrofitted with Bridge Rail Retrofit, Type 1, 2, or 3.

Type 1—Thrie beam rail with timber blocks anchored through the existing parapet. Type 1 should be used where the sidewalk is 18 inches [460 mm] or less in width.

Type 2—Thrie beam rail with steel posts anchored into the sidewalk. Type 2 should be used where the sidewalk is wider than 18 inches [460 mm].

Type 3—Concrete block anchored into the sidewalk. Designer must consider increased dead load. If weight is a problem, use Type 1 or 2.

If the existing bridge deck has sufficient strength, the designer may consider removing the existing railing and replacing it with a new concrete safety shape.

Timber railings are used mainly on timber bridges.

Approach railings are fastened to the bridge barriers and end with an approved end treatment. Impact attenuators are not used at approach railing terminations. Refer to DelDOT *Standard Construction Details* for details of barriers and railings, as well as for retrofit details. Also refer to Chapter Ten in the *Road Design Manual* and the *AASHTO Roadside Design Guide*.

9.5 JOINT REPAIR/REPLACEMENT

The types of joint devices common to Delaware bridges are:

- compression seal,
- strip seals,
- sliding plates,
- finger joints with troughs,
- open joints,
- hot-poured sealer joints,
- closed-cell elastomeric sealed joints,
- modular systems,
- elastomeric concrete joints, and
- silicone seals.

The most common shortcoming of joints is leakage. Leakage of roadway runoff containing chemicals, salts and pollutants causes rapid premature deterioration of decks, beam ends, bearings, and substructures.

The designer should always seal the deck joint over expansion bearings and control roadway runoff water. The Department installs sealed replacement joints during any deck rehabilitation, overlay, or replacement.

Usually, strip seal joints are used for rehabilitation projects. Strip seal joints are used for both transverse and longitudinal joints. Transverse strip seals are used for span movements up to 4 inches [100 mm]. Longitudinal strip seals are limited to 2 inches [50 mm] because of the hazard to motorcyclists.

Some existing bridges have open joints. Where these joints cannot be sealed or replaced during rehabilitation, the trough should be replaced, or if none exists, one should be installed to control runoff.

On long-span or curved bridges, finger joints or modular joints are used to accommodate movement. Finger joints are preferred. Troughs are required to control leakage of finger joints (see Figure 5-10). The other alternative, a modular joint, consists of three elements: the seals, separator beams, and supports. Proper function of the modular joint relies on equal seal deformations. The use of modular joints requires approval of the Bridge Design Engineer.

The designer is alerted to exercise care when computing joint openings and designing skewed joints. Refer to Figure 5-9 for sample calculations for strip seal joints.

9.6 APPROACH SLABS

The primary failures of approach slabs include undermining, settlement, and cracking.

Undermining. Undermining of bridge approach slabs is the result of the erosive action of water, scouring and carrying away material from beneath the slab. Damage to the slab caused by slab undermining can be repaired by pressure grouting. These methods fill the voids beneath the slab. The cause of the undermining is usually leaking joints. The joints must be resealed to prevent recurrence. In extreme cases, approach slab undermining can result in cracking and displacement of the slab, requiring replacement.

Settlement. Settlement may be repaired by pumping material to fill the voids under the slab or to raise the slab. These materials include:

- cement grout,
- flowable fill, and
- expansive polyurethane.

Cement grout may be applied under sufficient pressure to raise the slab. Flowable fill will fill the voids, provide support for the slab, and reduce lateral pressure on the substructure. Flowable fill must be placed in lifts and allowed to cure between lifts to prevent excessive load on the abutment. Expansive polyurethane is applied in liquid form. It expands about 30 times in volume as it solidifies. It seals voids in addition to correcting the slab profile. The application technique is proprietary and should be used only by experienced contractors. An overlay may be necessary to restore the ride with any of the above methods.

Cracking. Small cracks can be repaired by the application of low-viscosity crack sealer. If the cracks are too wide to permit sealing, partial or complete removal and replacement of the slab may be necessary.

If the approach slab has been previously overlaid with bituminous concrete, replacement with Portland cement concrete should be considered so the bituminous overlay is not needed. This should be done in conjunction with deck overlay or replacement.

9.7 BEAMS AND GIRDERS

9.7.1 CONVENTIONALLY REINFORCED CONCRETE BEAMS

In addition to visual inspection, the primary tests used to evaluate beams are sounding and coring. The tests and evaluation options for conventionally reinforced concrete beams and concrete-encased steel beams are the same as those for concrete decks. Refer to Section 9.3.1.

In addition to the five factors (i.e. spalling, scaling, cracking, efflorescence, and ASR) the designer must evaluate each beam or girder for collision and fire damage.

9.7.1.1 Concrete Beam Repairs

There are numerous repair methodologies available for repairing concrete beams. These include various materials, techniques, and application methods. The designer must evaluate the structural effects of the repair—particularly the effects of concrete removal—on the capacity of the member being repaired. A structural analysis may be necessary. A pigmented waterproofing sealer should be applied to the entire beam to create a uniform appearance.

9.7.1.2 Repair Materials

Potential repair materials include:

- cement-based mortar or concrete,
- nonshrink quick-setting mortar,
- epoxy mortar,
- resin-based polymer concrete,
- cement-based polymer concrete, and
- pneumatically applied mortar.

Factors to be considered in selecting repair materials include:

- size,
- location,
- general function of the member, and
- portions to be repaired.

Material selection is influenced by:

- compatibility of the material with the existing concrete,
- environmental considerations, including aesthetics,
- cost effectiveness,
- expected service life,
- availability of the material, and
- familiarity of contractors with the material.

Properties of the repair material should conform as closely as possible with the existing concrete, particularly with respect to the coefficient of expansion and modulus of elasticity. This physical compatibility is necessary for a good bond to the concrete substrate. Strength should be at least as high but not significantly higher than that of the existing concrete.

Repair material should have low shrinkage, low permeability, and low water-cement ratio to prevent moisture and chloride penetration. It should not react chemically with the embedded steel.

Repair material should adhere to the concrete surface, by applying either a rich cement mix or an epoxy bonding compound to the prepared concrete surface before placing the new material.

The portions of beams subject to water damage should be waterproofed.

9.7.1.3 Surface Preparation

Obtaining bond between the repair material and the existing concrete is extremely important in any concrete repair. The existing concrete must be clean and sound. The edges of the repair area must be saw-cut 1 inch [25 mm] deep into squares or rectangles. Saw cuts should be stopped at the corners to prevent overcutting. The corners must be hand chipped. Removal of concrete within the saw-cut area may be performed by jack hammering, hydrodemolition, or hand chipping. After removal, the remaining concrete surface should be thoroughly cleaned of loose concrete using high-pressure air or water. The reinforcing steel is sand blasted until completely free from corrosion. If the reinforcing bar serves only as temperature or distribution reinforcement, up to 30 percent section loss can be tolerated; only 10 percent section loss is permissible for primary reinforcement, and then only if analysis supports this reduction.

Steel corroded beyond acceptable levels normally requires supplemental reinforcement. Enough concrete should be removed to provide adequate lapping of bars beyond the deteriorated location.

Mechanical splices are preferred over lap splicing.

9.7.1.4 Surface Patching

All concrete repair patching requires the use of an approved bonding agent that bonds new and existing concrete. Generally, the Department uses epoxy- or water-based bonding agents.

Surface or shallow patching is usually limited to placement of repair material between the surface and the first mat of reinforcing steel. If the repair is deeper, refer to Section 9.7.1.5. Both shallow and deep spall repairs are illustrated in Figure 9-5.

Cement-based mortar or concrete, nonshrink quick-setting mortar, and epoxy mortar are suitable for surface patching.

Cement-based mortar or concrete is widely available, low cost, and compatible with existing concrete. Cement mortars are used for small thin repairs, and concrete is used for larger repairs (greater than 2 ft^3 [0.056 m^3] and greater than 3 inches [75 mm] deep). Trowel-grade cement-based mortars can be applied to vertical and overhead surfaces, while concrete must be formed.

Nonshrink quick-setting mortar is made with special admixtures or high early strength cement. The nonshrink characteristic is achieved by the use of expansive cements. They are suitable for the same repairs as cement-based mortar or concrete. To reduce costs, they can be mixed with up to 50 percent of their weight of pea gravel with no significant reduction in performance.

Epoxy mortar is a high-cost material and is used only where other materials are unsuitable. It is composed of three components: epoxy resin, a curing agent, and aggregate. It has high compressive and shear strengths, and excellent bonding properties. Epoxy mortars are very quick-setting and are not suitable for large-volume repairs. They are appropriate for patches up to 0.5 inch [13 mm] deep. Epoxy mortars are also effective for grouting anchor bolts and repairing bearing pads.

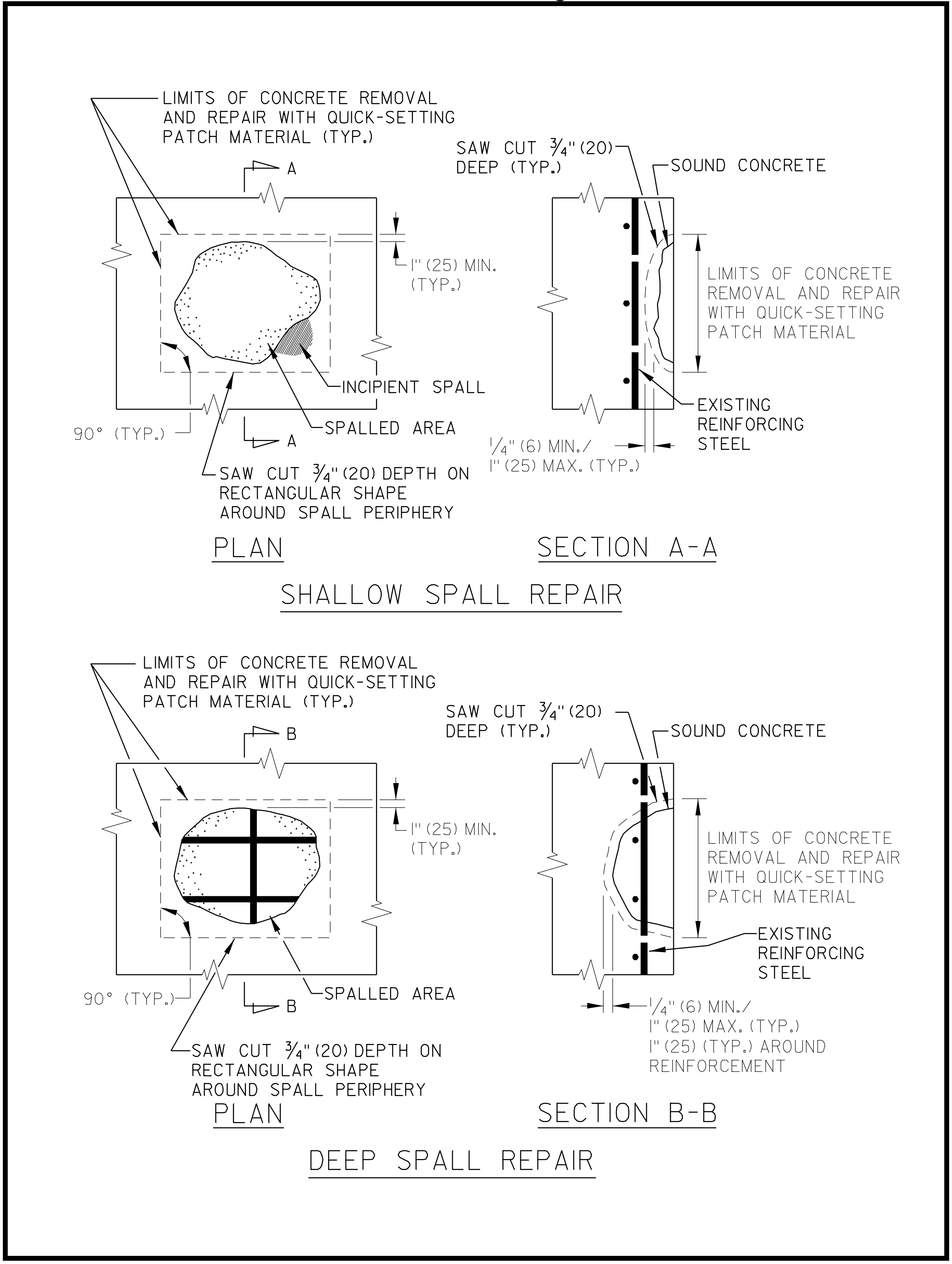
9.7.1.5 Deep Repairs

Deteriorated concrete that is too deep for surface patching and extends deeper than the first mat of reinforcing steel requires deep repairs. Preparation is the same as previously stated, except that the first mat of reinforcing steel is completely exposed to a minimum of 1 inch [25 mm] behind the reinforcement. These bars are thoroughly cleaned, in accordance with Section 9.7.1.3. (See Figure 9-5.) A suitable bonding agent is applied for any of the following repair materials.

Cement-based cast-in-place concrete is used for the repair of deteriorated areas 3 inches [75 mm] deep or more.

Resin-based polymer concrete is a methyl methacrylate monomer mixed with aggregates. They gain very high strengths in a short period of time, which may be unsuitable for some locations. In selecting this material, the designer should consider the high vapor pressure, low flash point, and intolerable odor. The volume can be increased by the inclusion of dried aggregate. It is suitable for locations where it can be placed quickly after mixing before it sets up.

Figure 9-5
Concrete Beam Patching Details



Cement-based polymer concrete is a mixture of polymers with cement-based mortar. The mixture enhances the mortar's physical properties in the presence of water. (One of the best-known cement-based polymers is latex-modified concrete.) It should be avoided for large-volume repairs because of the high cost.

9.7.1.6 Pneumatically Placed Concrete

Pneumatically placed concrete, such as shotcrete, or gunnite, is a mixture of sand, cement, water and admixtures, if appropriate. The mixture is sprayed at high velocity onto concrete surfaces. A bonding agent is normally applied prior to application of the mortar. Pneumatically placed concrete, up to 3 inches [75 mm] in depth, may be used for repairs to concrete structures or members. This material is also suitable for large-volume, cosmetic repairs. Pneumatically applied mortar works well for overhead applications.

9.7.1.7 Pressure Injection

Cracks in concrete are caused by:

- shrinkage,
- excessive thermal stresses where relief joints are not provided,
- improper concrete placement or curing,
- uneven foundation settlements,
- overstressing caused by internal or external forces,
- inadequate reinforcement,
- improper detailing, and
- corrosion of reinforcement.

Some cracks are active, and others are not. It is important that the cause of the crack be identified before attempting to seal or repair it.

Pressure injection of cracks is an acceptable repair method. Cracks up to a width of 0.25 in [6 mm] can be successfully filled with epoxy grouts of varying viscosity. Cracks wider than 0.25 in [6 mm] can be filled with either a cement grout or an epoxy grout that includes filler materials. Cracks less than 0.125 in [3 mm] in width can be surface sealed for corrosion protection or pressure injected with an epoxy resin when strengthening is necessary.

Details on methods, techniques, and the mechanics of crack injection can be found in *Bridge Inspection and Rehabilitation: A Practical Guide*, by Parsons Brinckerhoff.

9.7.2 PRESTRESSED CONCRETE BEAMS

By definition, prestressed concrete members are compressed by the induced tension in the prestressing strands. Significant concrete deterioration or corrosion of strands can lead to a loss of prestress force. Cracks and other deficiencies in prestressed concrete must be carefully investigated through a thorough analysis. Additionally, impact damage from overheight vehicles can also introduce severe damage that necessitates repair.

9.7.2.1 Damage Evaluation

Generally, a visual inspection of prestressed concrete beams is necessary to evaluate beams. Obvious spalls, impact damage, cracks, and rust staining should be noted. Certain types of defects are not as readily obvious.

- **Corrosion of prestressing strands**. Most corrosion of prestressed strands can be attributed to chloride attack. Leaking decks and joints and cracked girders expose prestressing strands to moisture. Often, the moisture contains chloride. Tests should be conducted to determine chloride contamination; electrical potential (half-cell tests) for steel corrosion; concrete cover; and concrete strength, composition, quality, and permeability. Where strands are unbonded, the chances of corrosion increase dramatically. Water leaking through the anchorage travels along the strand, causing it to corrode.

- **Cracking between adjacent precast box beams**. Where precast, prestressed concrete box beams are adjacent to each other, longitudinal cracks in the deck or wearing surface normally develop along the joints between the box beams. Cracks develop for the following reasons:
  - Transverse prestressing force induced by the tie rods typically fails to produce a uniform and integral behavior of all units.
  - Grouted shear keys between adjacent boxes may not have sufficient strength.
  - The concrete's shear capacity is inadequate without an overlay.
  - Variation in concrete creep may produce a difference in the camber growth in the box beams, generating forces that contribute to longitudinal cracking.
  - Shearing forces, created by the tendency of each box beam to deflect independently under truck loads, may cause cracking of shear keys.
  - Corrosion in the tie rods may produce a loss of lateral force.
- **Cracking of precast box beams**. If structural cracking develops in the bottom flange of precast box beams, the cracks may be sealed. This repair is unlikely to restore the structural capacity to its original value. An analysis should be made to evaluate the capacity of the superstructure in the cracked condition. The results of this analysis will determine the need to replace the superstructure.

  Water that collects inside a box beam increases the dead load and causes deterioration. Drain holes must be provided in precast box beams to allow any water that enters through cracks to escape. During the visual inspection, a check should be made to determine whether the drain holes are open. Provisions to unplug drain holes may be needed.
- **Damage from collision**. In most cases, vehicular collision is the cause of concrete loss and strand exposure over traffic lanes. Exposure of strands poses no immediate danger to the integrity of the beam unless there is a substantial loss of concrete. The concrete spall can be repaired after the strands are cleaned.

  A nick in one or several wires of seven-wire strand may not be serious. Severed or sharply bent wires in a single strand may not reduce beam capacity substantially. An analysis, which includes a fatigue evaluation, should be performed to determine whether strength loss is significant.

  Severance of more than two strands is considered a cause for concern and requires strand repairs, splices, or beam strengthening by post-tensioning. Traffic should be detoured from the area immediately over the beam, and the beam should be supported temporarily.

9.7.2.2 Repairs

There are several repair methods available, including the following:

- **Post-tensioning**. Refer to Figure 9-6. One method is the placement of symmetrical jacking corbels on either side of the damaged area, in the sound sections of the beam, and anchoring them to the bottom flange. Post-tensioning tendons are passed through the corbels and anchored against the bearing plates. After preloading the beam, the concrete is repaired and allowed to harden; when the beam is strong enough, the preloading is removed and the exterior post-tensioning of the beam is applied by turning the nuts at the ends of the threaded bars, simultaneously at both corbels. Threaded bars are normally placed in plastic conduits and pressure grouted. Intermediate supports should be provided for long bars.

  Jacking corbels should be located in an area where holes can be drilled through the web without interfering with either straight or draped strands. Shear transfer is obtained by roughening the area of contact between the corbels and the beam, and by installing expansion bolts at frequent spacings.

- **Internal splicing**. An efficient method of repairing a 0.5 inch [13 mm] strand is shown in Figure 9-7. The splice is torqued to induce a tension in the strand equal to that of adjacent undamaged strands.

- **Metal sleeve splice**. This is an external procedure for splicing a damaged beam. It does not normally restore prestress, although partial or full prestress may be restored by preloading. Where there are many severed strands, or where a large quantity of concrete is missing, this splice is often used to restore the beam to its function.

  Construction normally includes applying the necessary preloading, replacing the concrete, removing the preloading after the concrete has gained sufficient strength, and installing the metal sleeve. See Figure 9-8.

- **Composite Strips**. This is a reinforcing procedure for concrete surfaces. The material for reinforcing strips is either carbon fiber or E-glass fiber. The strips are attached to the sound concrete surface using polyester resin or an approved adhesive. The strips are used to restore or increase the load-carrying capacity of concrete members.

9.7.2.3 Preloading

Preloading is the application of a temporary vertical load during the in-place repair of a damaged prestressed member. Preloading reduces the stress in the concrete or strands in the damaged area during repairs. Preloading can be applied by either a loaded vehicle or vertical jacking depending on what is to be repaired. Preloading force requirements must be specified in the contract documents.

Figure 9-6
Post-Tensioning Method

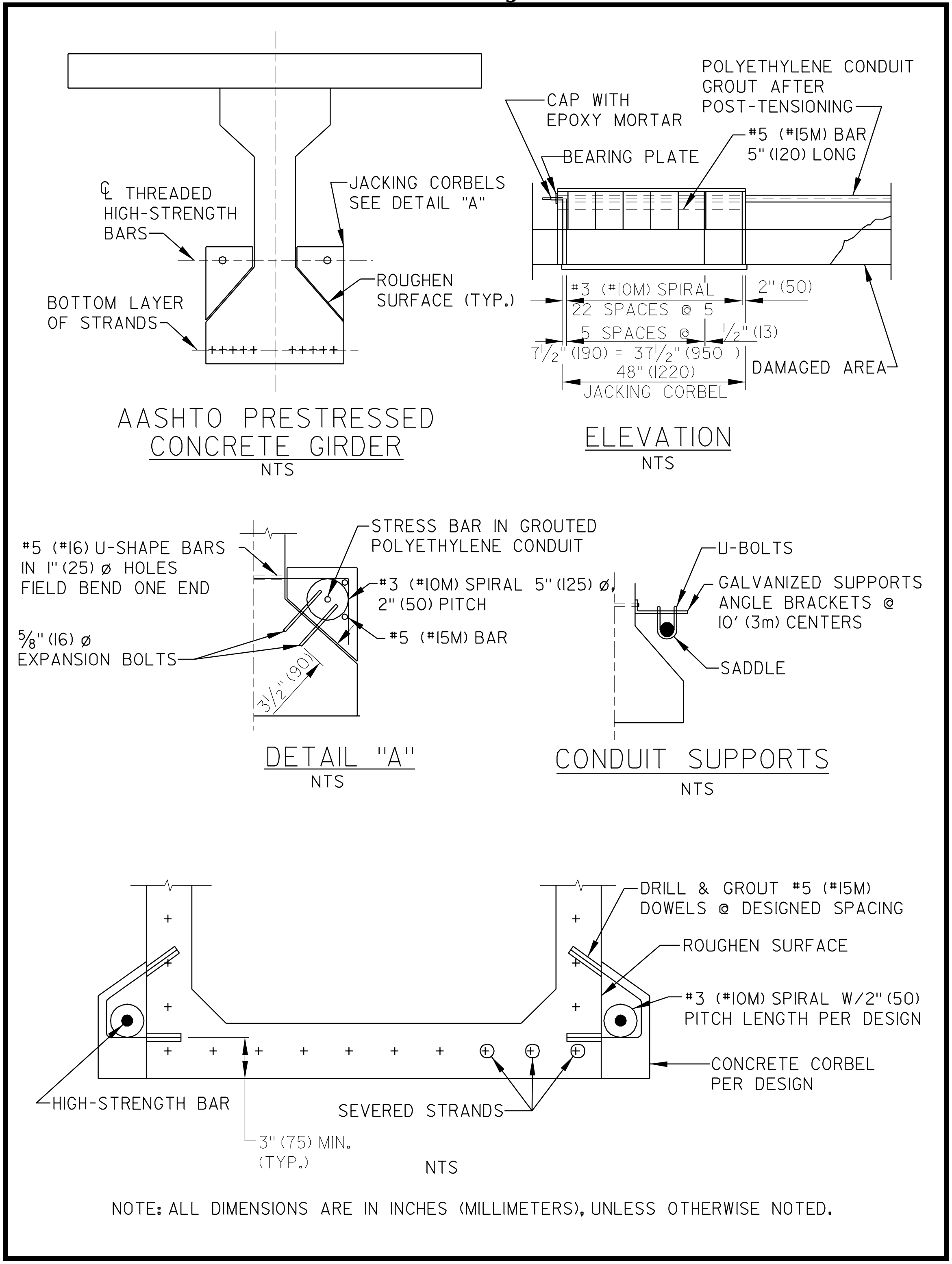


Figure 9-7
Internal Splicing

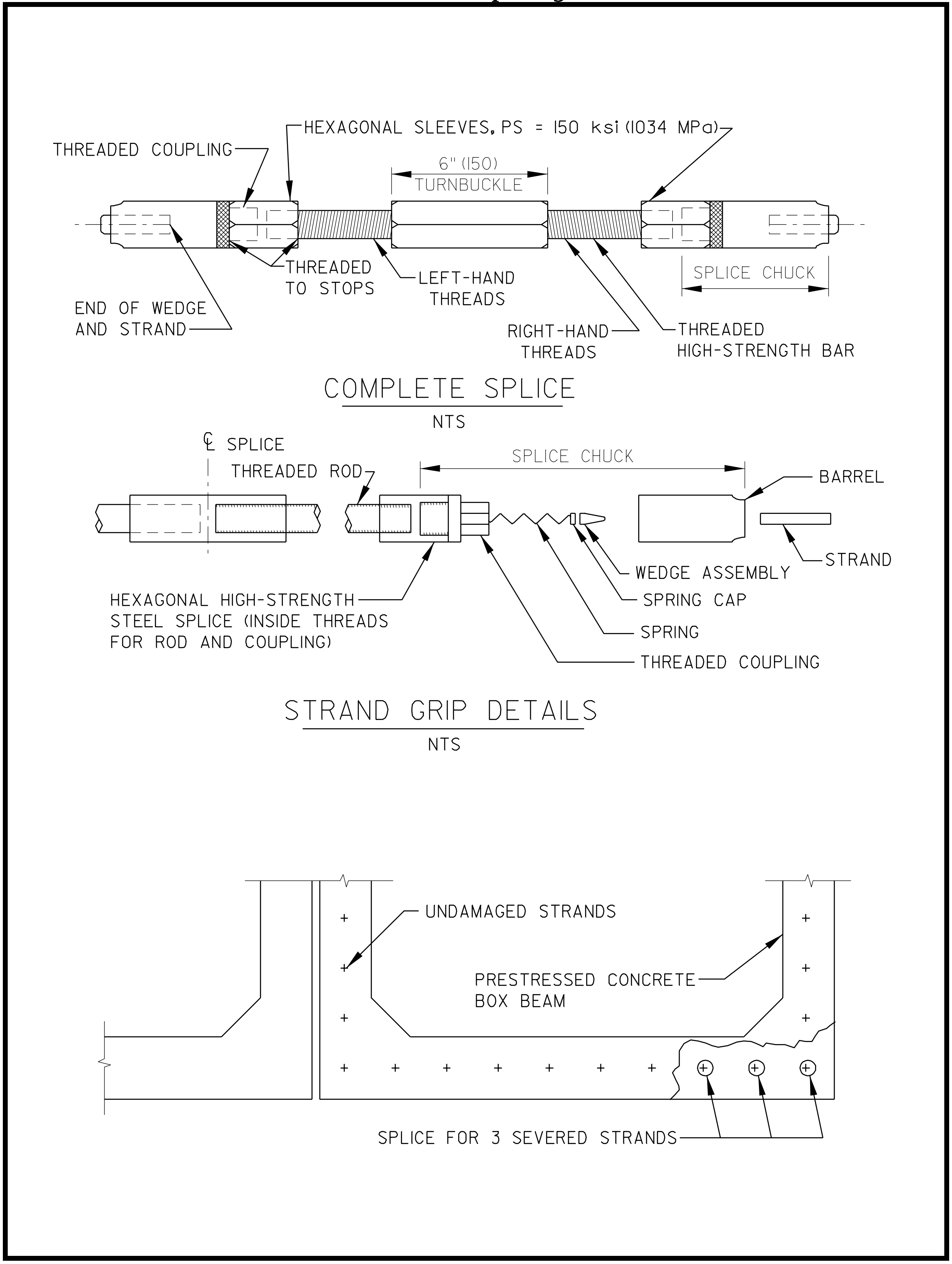
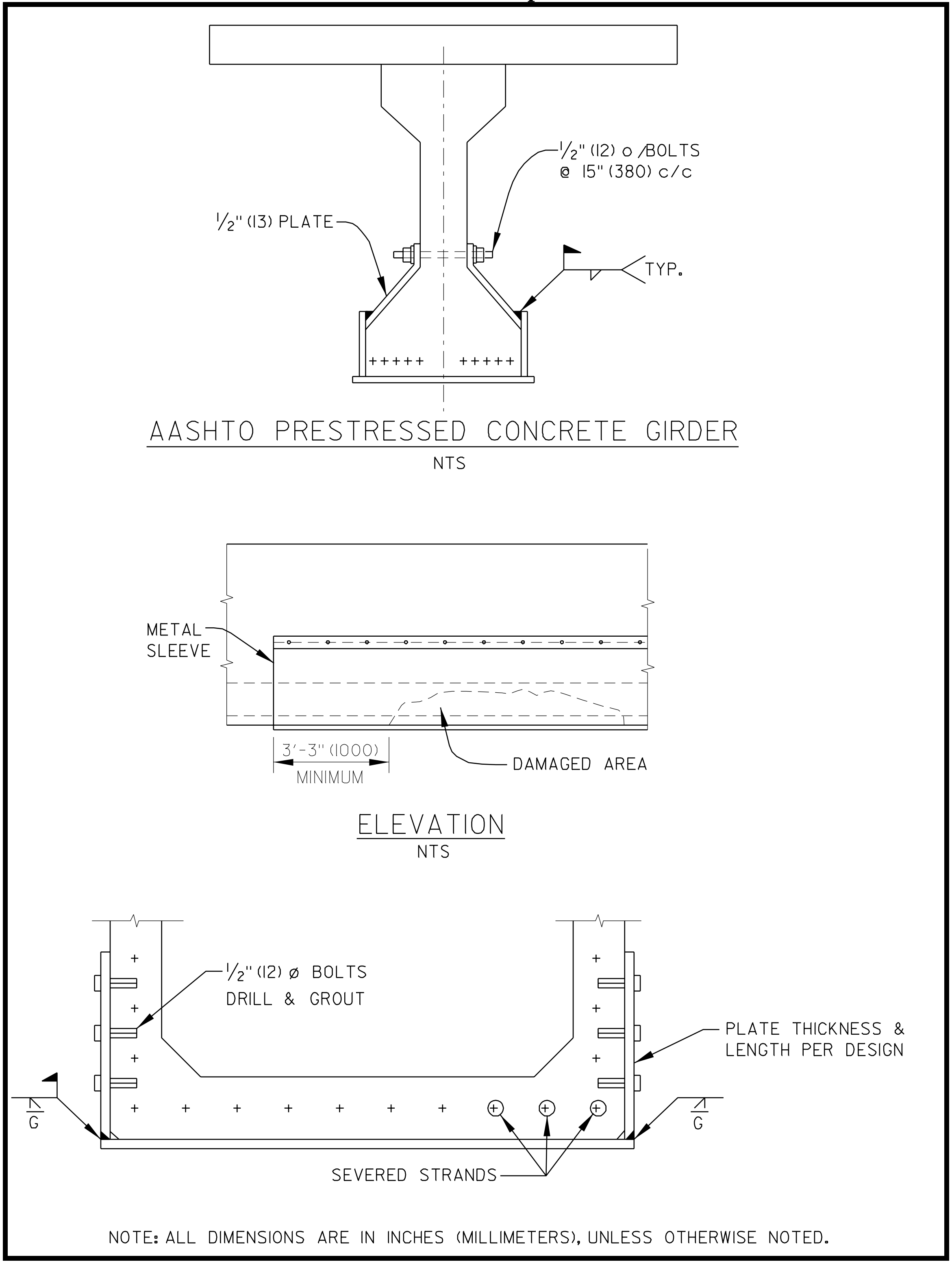


Figure 9-8
Metal Sleeve Splice



Concrete repair. Where a significant portion of the concrete has been lost, without any loss to the prestressing strands, the beam can be restored to its original condition by preloading without adding prestress and repairing the concrete. The amount of preload is determined by computing the loads that must be applied to bring the beam's concrete stresses within allowable limits. In a simple beam, maximum compressive stress caused by prestressing occurs at midspan under dead loads. Because of loss of concrete in the bottom flange, compressive stresses may exceed the allowable limit.

Preloading the beam using a loaded vehicle will reduce compressive stresses of the bottom flange and stretch out the strands in the damaged areas. Placing repair concrete and allowing it to cure before removing the preload restores the prestress force in the repair area and induces compression, thus enhancing durability and performance.

Strand repair. Where several strands have been damaged or broken, the beam can be restored to its original condition by preloading, repairing the strands, and replacing the concrete. The amount of preload is determined by computing the loads that must be applied to bring the beam's strand stresses within allowable limits. In a simple beam, maximum strand stress occurs at midspan under live loads. Because of the damage to the strands, strand stresses may exceed the allowable limit. Preloading the beam by jacking will reduce strand stresses. Care should be taken to avoid cracking of the remaining beams by excessive preloading. Strands are then repaired by splicing or another method. Repair concrete is then placed and allowed to cure. When the concrete has cured, the preload is removed and the beam restored to service. Application of a concrete sealer will enhance the durability of the repair.

9.7.2.4 Beam Replacement

If a beam cannot be repaired, it must be replaced. Replacement of a single prestressed concrete beam in a multi-beam prestressed concrete span requires consideration of the actual stress of the remaining beams, including the long-term effects of creep and tendon relaxation. It is not possible to replace one beam in an adjacent box beam superstructure.

The new member design must closely match the existing member capacity once the new sections of the deck are in place. Refer to Section 5, Concrete Structures, of the *AASHTO Specifications* for design criteria and procedures.

9.7.3 STEEL BEAMS AND GIRDERS

9.7.3.1 In-Depth Investigation

As with other structural members, an in-depth investigation must first be conducted to evaluate the repairs needed for steel beams and girders. The following must be evaluated:

- extent and location of corrosion,
- presence of cracking,
- collision or fire damage,
- fatigue stress and fatigue life analysis, and
- structural capacity.

Additionally, steel rehabilitation must include environmental considerations as well as structural considerations. The existence of fracture-critical members requires the designer to analyze the capacity and closely scrutinize the condition of each member.

In addition to specific defects, the designer must consider such information as age, capacity, remaining life, and loading. Each of these affects the economics involved in deciding whether to repair or replace a member.

9.7.3.2 Corrosion

The designer must determine the extent of section loss of each steel member due to corrosion. Corrosion-induced section loss must be measured and included in an analysis. Both the location and extent of section loss must be defined and included in any calculations.

9.7.3.3 Brittle Fracture

For a brittle fracture to occur, three conditions are necessary:

- lack of ductility or material toughness,
- stress, and
- a drop in temperature.

The fracture resistance of a structure depends primarily on material toughness and the ability to redistribute loads to other bridge components. Designers should be cognizant of these aspects, particularly as they relate to fracture critical members.

9.7.3.4 Fatigue Cracks

The fatigue strength of a member is affected by welds, holes, notches, loss of section, corrosion, and pitting. These can lead to cracks and fracture of a member resulting from crack propagation leading to a bridge failure. Cracking can be detected visually, provided the crack is large enough to be seen and is not covered by paint, rust, or debris. Smaller cracks can be found by nondestructive methods, such as dye penetrant, magnetic particle, ultrasonics, and X-ray techniques.

Areas of concern usually occur in welded members at the following locations:

- cover plate terminations,
- flange and butt splices,
- changes in section,
- stiffener end welds, and
- lateral bracing.

For riveted or bolted members, cracks are typically found at the following locations:

- flange angle rivets,
- cover plate terminations,
- floor beam ends, and
- gusset plate connections.

A fatigue analysis must be conducted to determine the live load stress range in a member versus the allowable. Refer to Section 6.6, Fatigue and Fracture Considerations, in the *AASHTO Specifications* for fatigue analysis criteria and procedures.

9.7.3.5 Fire Damage

Exposed portions of steel bridges subjected to temperatures above 1140^{o} F [615 C] (evidenced by damage to the zinc or lead primer) will suffer plastic deformations by exceeding the yield strength or buckling caused by stresses beyond the limit of elastic stability. The degree of damage will depend on the maximum temperature to which the steel was exposed, the duration of the exposure, and the member loading.

Embrittlement of the steel member can occur from very high temperature exposures. In cases where the steel has been exposed to very high temperatures with significant deformation resulting, the members should be replaced. Heat straightening will not work in this case.

9.7.3.6 Collision Damage

The designer should refer to *NCHRP Report 271, Guidelines for Evaluation and Repair of Damaged Steel Bridge Members*.

9.7.4 REMEDIAL MEASURES

The appropriate type of bridge remedial measure depends on the specific bridge being rehabilitated. Different structures pose different requirements on the designer. Structures should be evaluated on a case-by-case basis.

Important to any remedial design is the weldability of the steel. This can be determined by sampling the steel in question and performing a chemical as well as a standard yield test of the “coupon” sample. The specifications defining the requirements for strength testing and chemical analysis are covered by the American Society for Testing and Materials in ASTM A370-90a and ASTM A751-90 respectively. AASHTO specification T-244-90 for mechanical tests corresponds to ASTM A370-90a, but there is no corresponding chemical test specification. The coupon must be of sufficient size to provide the machined specimens described in the ASTM test procedure.

Possible remedial measures include both preventive and reactive steps:

- cleaning and painting,
- cathodic protection,
- strengthening,
- crack repair,
- peening,
- bolting splice plates,
- heat straightening, and
- member replacement.

9.7.4.1 Cleaning and Painting

The extent of remediation of the coating of structural steel will depend on the condition of the existing coating. The designer has the following options:

- minor cleaning and overcoating,
- partial cleaning and repainting, and
- full cleaning and repainting.

The first step is a visual inspection to evaluate the condition of the existing coating. If overcoating is being considered, the designer should request the following tests from Materials and Research:

- adhesion (the higher the test results in psi [kPa], the better the adhesion),
- coating thickness for each existing layer, and
- compatibility between the new and existing coatings.

The designer must evaluate the test results in determining the best cleaning and painting options. Adhesion is measured by ASTM Test 4541, Test Method for Pull-Off Strength of Coatings Using Portable Adhesion-Testers. Structures must have test results above 200 psi [1400 kPa] to be suitable for overcoating. If the existing paint thickness is not greater than 20 mils [0.50 mm] and the adhesion test results are satisfactory, the bridge may be overcoated. If the paint thickness is greater than 20 mils [0.50 mm], the paint must be removed before the steel is repainted. A major consideration is the environmental requirements. Refer to Section 9.2. Other factors to be considered include:

- the age of the structure,
- whether cleaning will provide a surface suitable for painting,
- the condition of the existing coating and the extent of coating failure,
- the cost of removal and containment, and
- compatibility between cleaning methods and the specified coating.

If partial or zone painting is required, fascia beams should be completely recoated. One source of information to assist with the evaluation is the Society for Protective Coatings.

Cleaning. The method and extent of cleaning depends on the condition of the existing coating, the extent of repainting required, and the coating to be applied. Normally, the Department requires near white cleaning (SSPC-SP10 or SSPC SP11) for repainting existing steel. If overcoating is specified, a high-pressure water blast is used as surface preparation. Regardless of the extent, there are two primary areas of concern during the cleaning of steel structures - hazardous paint removal and airborne abrasive blast dust which escapes a bridge site during steel cleaning operations. Refer to Section 9.2.1 for environmental considerations. All blast residue must be contained and disposed of in accordance with environmental requirements. This is achieved by enclosing the structure being cleaned to collect the residue.

Painting. The following factors should be considered when selecting a coating system:

- compatibility of the proposed coating with the existing coating,
- the presence of airborne chemical fumes or volatile organic compounds,
- the presence of water spray or misty conditions caused by nearby water,
- the height of the members above flood or tidal levels,
- unusual roadway conditions (including open steel-grid decks) that allow drainage to pass over the structural steel or to pond water on the deck, and
- accumulation of snow, ice, or debris against steel surfaces.

Usually, the Department uses different painting requirements when full repainting or overcoating is required on rehabilitation projects as for new bridges. Refer to Section 5.3.5. Due to changing technology, the designer is encouraged to seek out the latest Department standards for paint application.

9.7.4.2 Cathodic Protection

Cathodic protection for steel members may be used only with the approval of the Bridge Design Engineer.

9.7.4.3 Strengthening

Section loss may cause sufficient reduction in member capacity to require strengthening the member.

Strengthening may be accomplished by bolting or welding additional members. Strengthening a portion of a structure can be accomplished by adding a new member or replacing an existing member with a new member with greater capacity. Other methods of strengthening that the designer may consider include:

- replacing an existing member with a stronger, shallower member to increase vertical underclearance,

- adding cover plates to the bottom flange of existing stringers, and
- adding new members to share loads with parallel existing members.

Cover plates should be bolted into place.

9.7.4.4 Crack Repair

Most cracks are caused by fatigue or accident damage. There are three important considerations necessary before a decision can be made to repair a crack in a steel member:

- the cause of the crack,
- the potential for growth of the crack, and
- the size of the crack.

Generally, visible cracks should be repaired. Common repair methods include:

- drilling a hole at the end of the crack,
- strengthening the area of the crack,
- replacing the member, and
- eliminating the cause of the crack.

Prior to drilling, use ultrasonic testing to determine the location of the end of the crack. Refer to FHWA publications *RD-89-166, Fatigue Cracking of Steel Bridge Structures, Volume I, RD-89-167, Fatigue Cracking of Steel Bridge Structures, Volume II*, and *RD-89-168, Fatigue Cracking of Steel Bridge Structures, Volume III,* for crack repair analysis information.

9.7.4.5 Peening

Peening is a cold working process in which the surface of the component is deformed either by a high-velocity stream of metal shot or by hammer peening. The objective is to create residual compressive stresses in the surface layer, thus reducing the severity of stress concentration. Peening should be considered where welds are used to repair existing steel members.

9.7.4.6 Bolting Splice Plates

Bolting may be used as a repair method or as a supplement to other repair methods. Replacing a damaged element with a new piece of steel fastened with high-strength bolts is regarded as the safest method of repair. Fracture-critical members and tension members should be repaired by bolting.

9.7.4.7 Heat Straightening

Heat straightening is a unique method used with jacking, blocking and supplemental supports to correct member misalignment caused by impact.

Not all accident-damaged members should be heat straightened. Some members cannot be straightened due to the extent of damage. With others, heat straightening may cause additional damage to the steel member, reducing member capacity. A member may be heat straightened only once at any one location on the member. Fracture-critical members must not be heat straightened unless they are supplemented by bolting splice plates over the straightened area.

Generally, a member is considered adequately straightened if it is returned to line, grade, and shape within 0.25 inch [7 mm]. Temporary support must be provided for beams while being heat straightened. Refer to *NCHRP Report No. 271, Guidelines for Evaluation and Repair of Damaged Steel Bridge Members*.

9.7.4.8 Member Replacement

Replacing a member will be necessary where deterioration, accident, fire damage, cracking, or overstress have caused such extensive damage or loss of capacity to make repair impractical and uneconomical.

When considering member replacement as a rehabilitation measure, the designer must perform rigorous analysis to ensure that the member can be removed and replaced without reducing the overall capacity of the structure. Temporary supports are generally needed unless the dead load is removed or supported by other temporary means.

9.7.5 BEARINGS

Bearings transmit loads to the beam seat area and accommodate movement of the bridge. Bearings are designed to carry static loads and dynamic forces. Bearings may be either fixed or expansion. Fixed bearings allow only rotation of the girders. Expansion bearings allow rotation and translation. The types of bearings commonly found in Delaware are:

- steel,
- bronze or steel sliding plate,
- rocker,
- elastomeric,
- polytetrafluoroethelyene (PTFE, also called Teflon®), and
- pot bearings.

Rocker bearings should be replaced with a more stable type.

All bearings shall provide full, uniform bearing when under full load.

9.7.5.1 Types of Problems

Any problem that prevents rotation or translation of a bearing should be corrected. There are five types of serious problems that require corrective action involving bridge bearings:

- deterioration and corrosion with or without section loss,
- bearing element misalignment in line with the member,
- frozen bearings,
- broken anchor bolts, and
- transverse displacement (especially on skewed bridges).

Generally, the primary cause of bearing problems is exposure of the bearings to the elements, combined with deck joint leakage, which carries rainwater, deicing chemicals, and dirt from the roadway above onto the bridge seat area.

9.7.5.2 Seismic Requirements

Bridges should be upgraded to meet current seismic requirements when they are rehabilitated. Refer to Section 5.5.2.

9.7.5.3 Bearing Rehabilitation Methods

The decision to rehabilitate or replace bearings depends on the current condition of the bearing and the potential for recurrence of the problem. Bearing rehabilitation methods include:

- cleaning and painting,
- jacking and resetting, and
- lubricating.

9.7.5.3.1 Cleaning and Painting

Extreme deterioration from corrosion can cause a bearing to lose section and, in the worst case, failure of the bearing to transmit loads to the substructure as designed. Routine maintenance cleaning (removal of accumulated debris) and cleaning and painting (power tool or blast cleaning and recoating) can protect and prevent serious section loss that might require bearing replacement. Cleaning and painting bearings is subject to the same environmental requirements as for steel beams. Refer to Section 9.7.4.1.

9.7.5.3.2 Jacking and Resetting

Jacking and resetting a bearing is usually required due to misalignment. Misalignment may occur on elastomeric, sliding plate, or rocker bearings. Jacking the beam can be accomplished by installing structural elements between beams and using hydraulic jacks to support and raise the beam. Refer to Section 9.8.3 for a discussion of jacking requirements.

Resetting bearings generally involves repositioning the masonry plate to align with the sole plate. This may require replacement of anchor bolts. If anchor bolts are not damaged or deteriorated, they can be reused.

Resetting bearings may introduce eccentricity of load into the substructure. All new bearing plates must be designed for the reorientation of loads. The designer should consider the new load path and the effects of jacking and resetting the new bearing plate on the substructure.

As a minimum, an analysis must be performed to review and check the original design.

A neoprene or fabric bearing pad with a minimum thickness of 0.125 inch [3 mm] should be placed under the new masonry plate to accommodate any bearing area roughness.

The bearing area and the area adjacent to the new bearing must not be deteriorated, and all surfaces must be sound and level.

Any condition that caused the bearing to be misaligned must be investigated and corrected.

9.7.5.3.3 Lubrication

Uncoated steel used in steel-on-steel expansion bearings will corrode. The corrosion will cause the bearing surfaces to become rough. The coefficient of friction between the plates will increase due to the corrosion, and the bearing will cease to function. This condition can be improved and avoided by lubricating the bearings or by providing bearing materials with low coefficients of friction. Bearings are either externally lubricated or self-lubricated through the selection of special materials.

The Department uses a low-viscosity penetrating grease applied externally to existing sliding plate or rocker bearings. Bearings are not jacked to place lubricant between the plates.

The function, roughness and condition of the bearing are the deciding considerations in replacement or rehabilitation of a bearing.

9.7.5.4 Bearing Replacement

Bearings are replaced as part of a bridge rehabilitation project, or when bearings or bearing areas become so severely deteriorated as to jeopardize structural safety.

Bearing replacement requires beam jacking. Jacking bearings can be accomplished by jacking from the bearing seat area or by constructing a jacking frame supported on the footing of the substructure or on soil under the bridge. When all bearings in a substructure unit are replaced, the replacement bearings should be upgraded to current Department standard bearings rather than the same bearing being replaced. Individual bearings are replaced in kind. The designer should consider seismic design in the analysis for bearing replacement. Refer to Section 5.5.2.

9.8 FOUNDATION AND SUBSTRUCTURE

Needed repairs to foundations and substructure units are generally included in any Department rehabilitation project. Types of rehabilitation efforts include:

- crack repair,
- replacement of deteriorated concrete,
- bearing seat repair,
- protection of substructure,
- pile repair,
- scour countermeasures,
- stabilization,
- underpinning,
- slope paving, and
- drainage.

9.8.1 CRACK REPAIR

Crack repair for foundations and abutments is similar to repair of cracks in concrete beams. Refer to Section 9.7.1.7 for repair methods.

A leaking crack can be pressure grouted if the flow of water is temporarily halted. This is done by inserting expanding chemicals into or behind the crack.

9.8.2 REPLACEMENT OF DETERIORATED CONCRETE

Replacement of deteriorated substructure concrete is accomplished using two methods: pneumatically applied mortar and cast-in-place. Methods that identify the areas of concrete removal include visual inspection, sounding, field chipping, and coring. Refer to 9.3.1.1. Chloride content and half-cell analyses are usually not performed on substructure components but can be considered.

Substructure repair methods, materials, and practices are the same as for repairs to concrete beams. Refer to Section 9.7.1.

9.8.3 BEARING SEAT REPAIR

Steel jacking beams, appropriately designed and detailed, are most suitable for jacking steel beam spans. Refer to Figures 9-9 for an example. Normally, the contractor is required to submit the jacking procedure for Department approval.

Jacking is required to accomplish bearing seat repairs. Bearing seat repairs may be required even if bearings are fully functional, such as where substructure concrete has spalled or deteriorated under a bearing masonry plate.

Jacking beams can be done from adjacent beam seat areas or from separate jacking frames. Jacking frames are used where jacking from the beam seat area is not practical. When jacking frames are used, subsurface investigation shall be performed. Cross-braced frames are best suited. Frames are founded on existing footings or temporary footings constructed on soil. Piles are not normally required under jacking frame footings because spread footings can provide sufficient bearing capacity. Where spread footings will not provide sufficient support, piles may be driven through holes in the deck. A plate or beam can then be welded to the pile (after it has been cut off to the required elevation) to support the jacking frame. Jacking frames are costly and should be avoided when possible. The designer must analyze the beam at the location where jacking of the beam will take place. Bearing and jacking plates, web stiffeners, and bracing may be required.

The jack must be positioned within the existing reinforced area of the beam seat at least 2 inches [50 mm] from the edges. This may preclude placing a jack in front of the bearing under the beam. Jack capacity must be specified with a factor of safety of 1.5 based on the calculated design jack load.

When individual bearings are being jacked, the load on the bearing should only be relieved. Raising the beam more than 0.125 inch [3 mm] may not be desirable. When all bearings on one end of a simple span are being jacked, lifting the beams is preferred. Lifting may cause deck cracking in continuous spans. In this case, the designer should only relieve the load. If traffic is being maintained on the bridge, the span should be raised only the minimum necessary to achieve the repairs. The deck joint configuration must be evaluated.

Manifolding of hydraulic equipment for jacking may be used, if approved by the Bridge Design Engineer. Because of jack variability, individually operated hydraulic jacks are better suited for jacking multiple beams or girders. Jacking under live load shall not be permitted unless otherwise approved by the Bridge Design Engineer.

Once a structure is jacked, the load must be secured before any existing material is removed. This can be done by two methods. Jacked loads are secured by either temporary blocking (short columns or cribbing), or the use of locknut jacks which are not dependent on hydraulic pressure. Hydraulic jacks are not used to support loads, even if the hydraulic pressure is maintained.

Lateral and vertical bearing movement should be monitored during any jacking operations. Standard surveying instruments should be used to detect undesirable movement. Provisions must be made in the design for expansion and contraction. Simple lubricated sliding plates may be effective. Stainless steel and TFE sheets between bearing plates can also be used.

Once the beams are jacked and blocked, concrete removal may begin. The concrete removal limits are first saw cut and the corners hand chipped. Saw cuts are stopped at the corners to prevent overcutting. Concrete is then removed. The limits should be squared and benched in both plan and section. Concrete is replaced in accordance with Section 9.7.1. Bearing seat repairs are performed using conventional reinforced concrete.

Figure 9-9a
Jacking Details

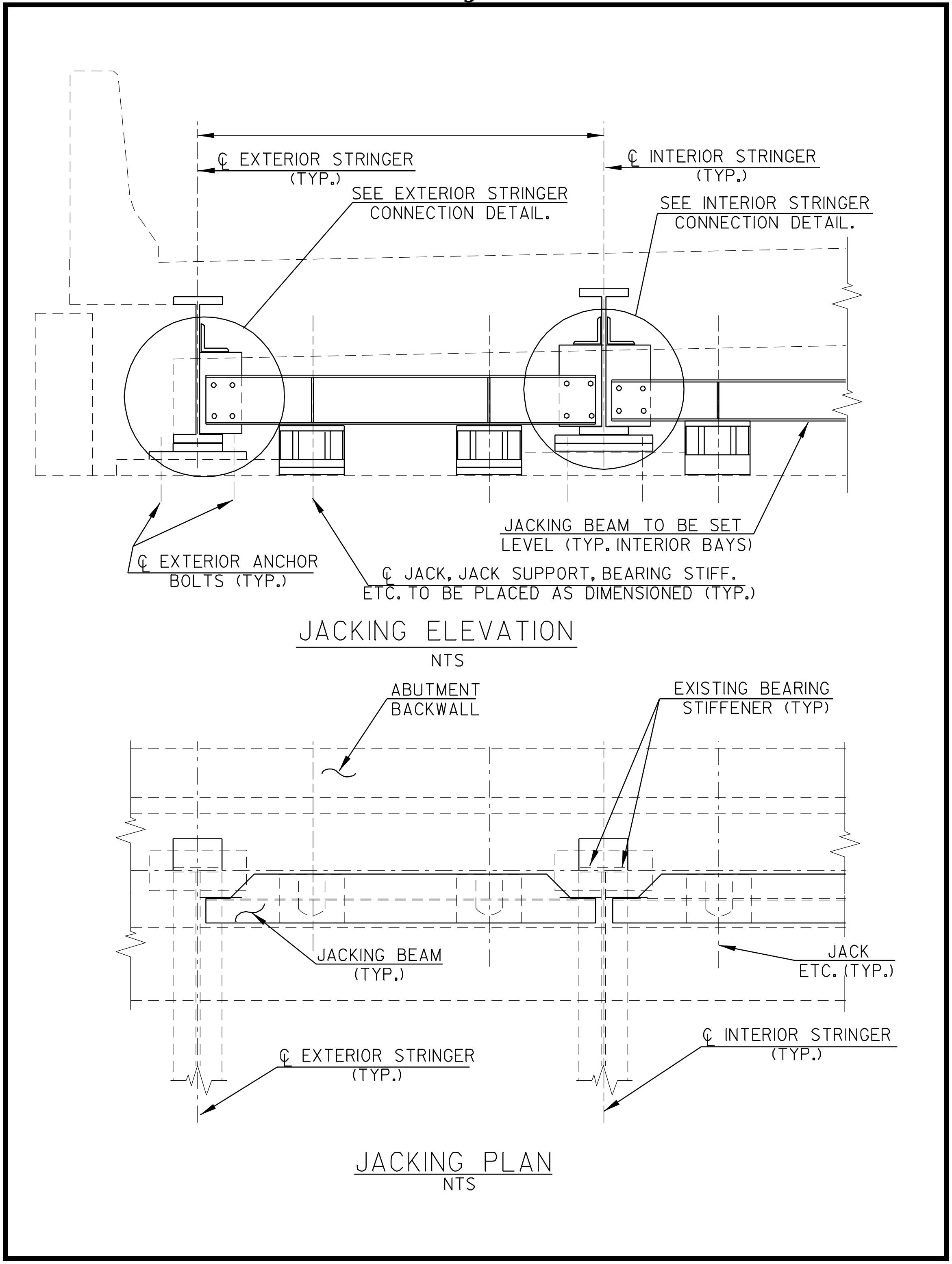
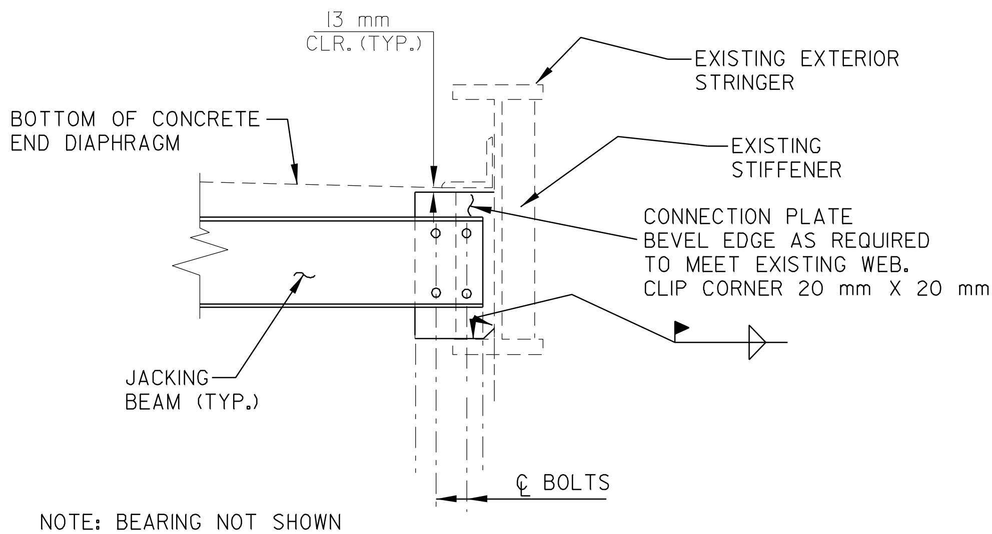
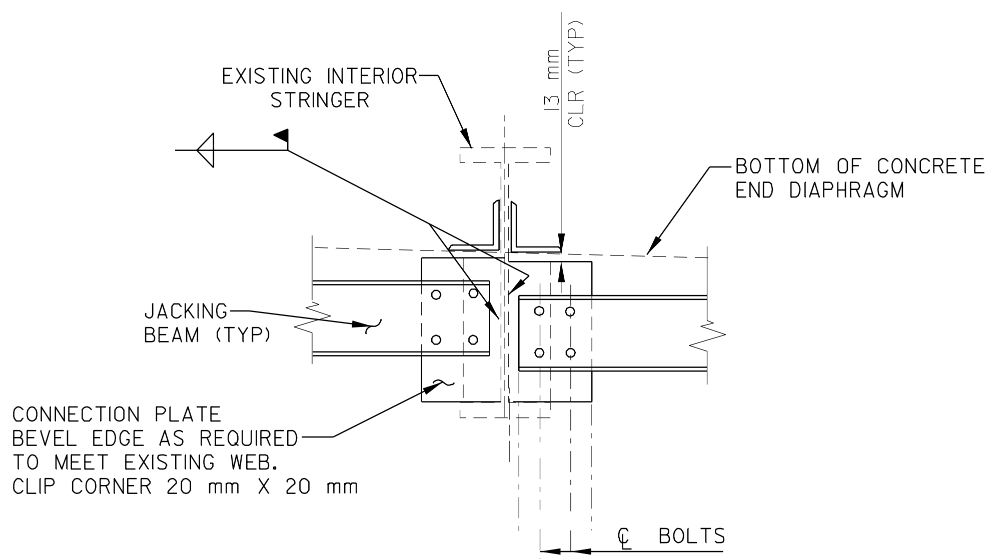


Figure 9-9b
Jacking Details



EXTERIOR STRINGER JACKING BEAM CONNECTION DETAIL

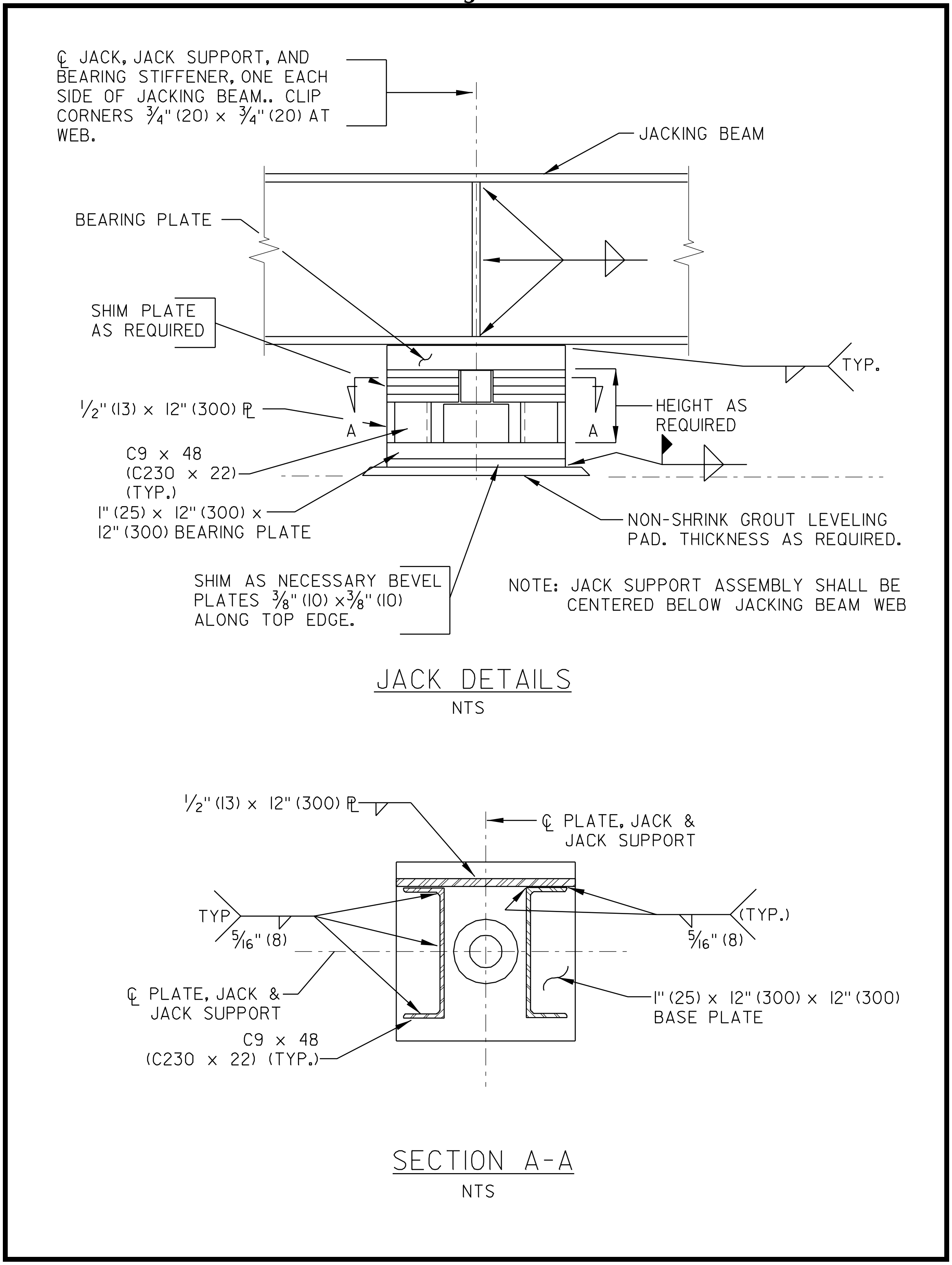
NUMBER OF BOLTS BASED ON DESIGN
NTS



INTERIOR STRINGER JACKING BEAM CONNECTION DETAIL

NUMBER OF BOLTS BASED ON DESIGN
NTS

Figure 9-9c
Jacking Details



After repairs are completed, the substructure bearing seat area should be power washed and the seat area, pier, or abutment faces and backwall sealed to prevent water penetration.

9.8.4 REPAIR OF CONCRETE SUBSTRUCTURES OVER WATER

Repair of substructure elements includes:

- underwater repairs,
- splash zone repairs, and
- above-water repairs.

9.8.4.1 Underwater Repairs

Small concrete spalls can be repaired underwater without dewatering using hand-applied epoxy mortar. Large repairs are made by forming the repair and placing conventional quick-setting concrete using tremie methods. In some instances, a diver will be required for inspection or placement of repair concrete under water. Where repairs must be made under water, the costs may be higher. If working in the dry is required, a cofferdam or similar method to create a watertight form must be constructed.

First, all unsound concrete is removed. The designer must ensure that the structural integrity of the substructure is not compromised in the removal operations. This includes individual elements as well as the integrity of the total substructure unit.

If reinforcing is encountered in the chipped area, it must be inspected and any deficiencies corrected. Any primary reinforcing bars with section loss greater than 10 percent should be supplemented. Temperature reinforcing can sustain up to 30 percent loss of section without being supplemented. Unreinforced sections should be reinforced by doweling 0.625 inch [16 mm] steel bars into adjacent sound areas. Center-to-center spacing of bars should not exceed 18 inches [460 mm].

A suitable form is then attached to the concrete using expansion bolts. The form may extend beyond the original concrete surface. A seal between the form and the concrete must be achieved. This can be accomplished with a gasket or other means. Refer to Figure 9-10.

Concrete is pumped into the formed void from the bottom and allowed to overflow the area out of an inspection hole at the highest point of the deteriorated area. More than one inspection hole may be needed. Sufficient excess concrete must be allowed to run from the inspection hole to insure the integrity of the repair area. After curing, the forms may be removed and the repair inspected.

9.8.4.2 Splash Zone Repairs

Splash zone repairs can be accomplished in the same manner as underwater repairs or, since the top of the repaired area is above the water surface level, the repairs can be accomplished “in-the-dry”. Refer to Figure 9-10.

Areas to be repaired in-the-dry are prepared the same as underwater repairs, except that once the form is installed and the seal achieved, all water is pumped from the void created by the chipped area and the form. Concrete is then placed using the tremie method, overflowing the form to remove any unsuitable or diluted concrete. Generally, the in-the-dry method requires the forms to be larger than the original concrete structure. After curing, the forms are removed or may remain as additional protection. Random inspection of the repaired areas is required, and in these cases removal of the forms is necessary.

Figure 9-10a
Substructure Repairs

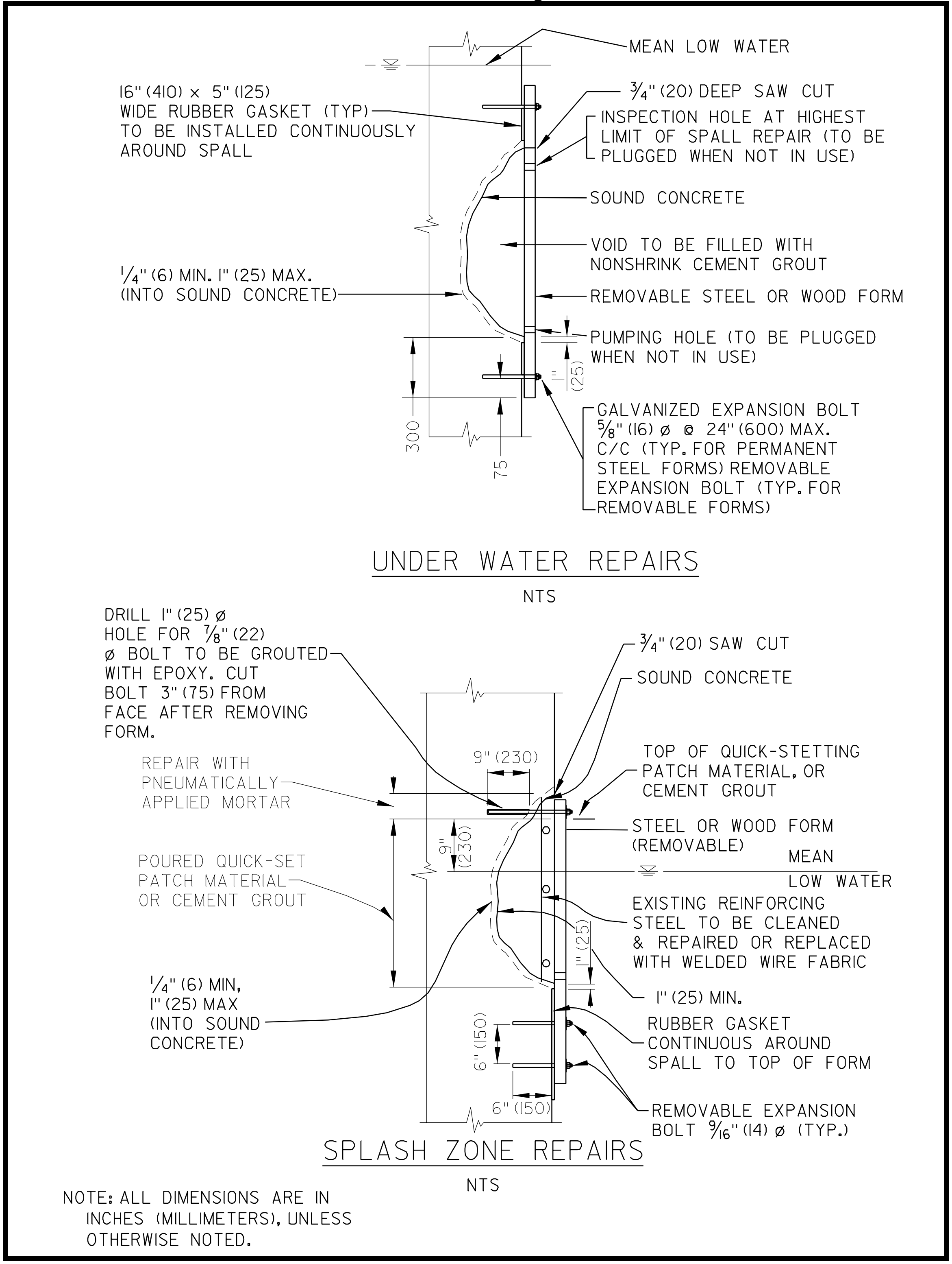
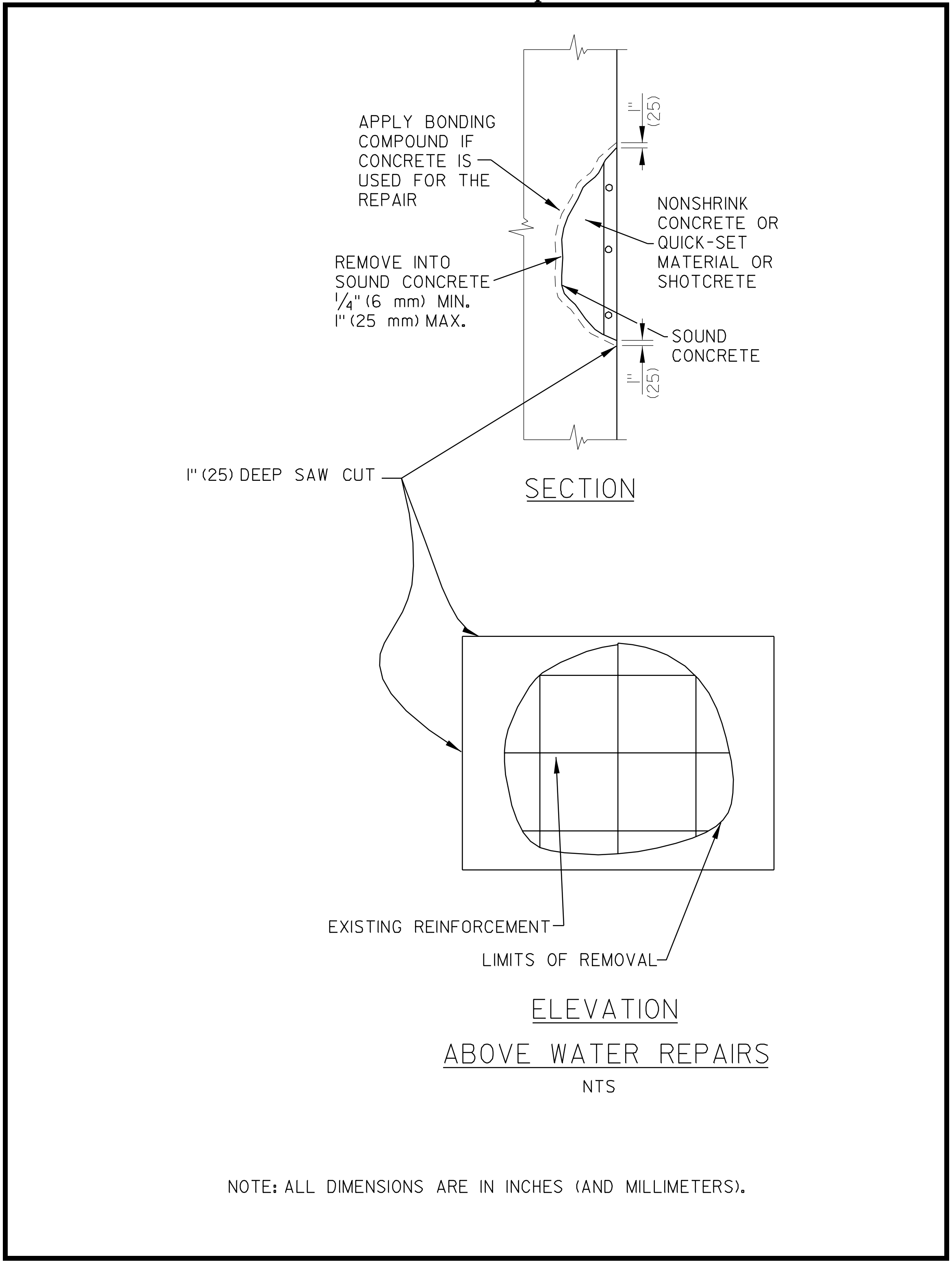


Figure 9-10b
Substructure Repairs



9.8.4.3 Above-Water Repairs

Above-water repairs to substructures are made with methods similar to those used to repair other concrete members. See Sections 9.3.1.4, 9.7.1 and 9.8.2, and Figure 9-10.

9.8.5 PILE REPAIRS

Two structural analyses should be performed to evaluate pile capacity: for the existing condition and after the removal of deteriorated concrete. The designer will evaluate the results of the analyses to determine any needed restrictions on live loads or contractor operations during construction. If the existing pile does not have adequate capacity to support dead load, supplemental support must be provided during repairs. See Sections 9.8.7.2 and 9.8.7.3 for methods of support.

One example of pile repairs for the splash zone is shown in Figure 9-11. This type of repair can be used for prestressed piles, CIP concrete piles, or H-piles. When significant deterioration of the pile has occurred, a detailed analysis must be performed to determine the repair requirements of the remaining section and the need for supplemental support during repairs.

- **Prestressed concrete piles**. If the analysis indicates inadequate capacity, replacement piles may be considered. If deterioration is not significant, encasement will protect the prestressed pile from further deterioration.
- **CIP concrete piles**. Where deterioration of CIP concrete piles has occurred, encasement, using the method shown in Figure 9-11, can adequately restore the section of the pile and the structural capacity. Severely deteriorated pile shells require close evaluation. Concrete deterioration within the shell may extend significantly beyond the splash zone. Repairs may require removal of all deteriorated concrete and restoration of pile capacity using supplemental members followed by encasement of the pile.
- **H-piles**. Section loss for H-piles must be repaired by adding plates, channels, or angles to the existing section before encasement. The supplemental members must extend beyond the limits of deterioration.

9.8.5.1 Timber Pile Rehabilitation

As with any material, repair or replacement of timber pile is eventually required. Repair methods used by DelDOT include the following:

- Concrete jacket
- Epoxy injection
- Reapplication of wood preservative
- Composite fiber wraps

Concrete jackets are used in areas of lost structural capacity due to rot, insect damage, or physical damage. The repair method consists of a fiberglass jacket 8 in [200 mm] larger in diameter than the pile, galvanized wire mesh and concrete fill. Details are shown in Figure 9-13.

Epoxy injection is carried out by installing injection ports, sealing the outside of the crack, and pumping the epoxy under pressure. The crack is considered full when epoxy flows out of the return ports.

Reapplication of wood preservatives is complicated because the common preservatives are hazardous and can be dangerous to humans and the environment in their liquid state. Therefore, DelDOT's practice for field preservative treatment is to drill holes and install solid "anti-fungal" cartridges. These cartridges contain a liquid anti-fungal wood preservative that spreads through the wood by capillary action. The drilled holes are then sealed with hardwood creosoted plugs.

Figure 9-11
Pile Repairs

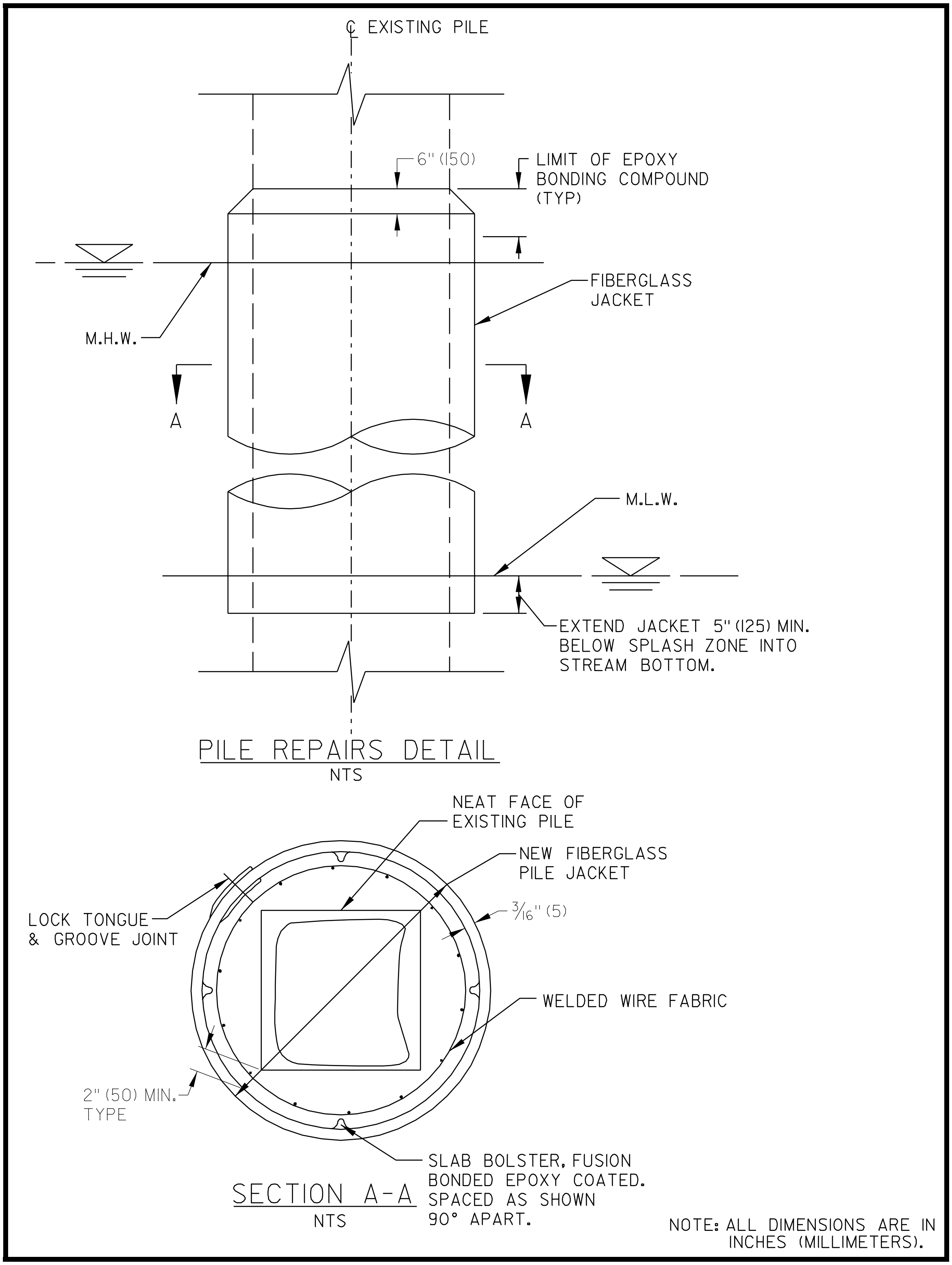
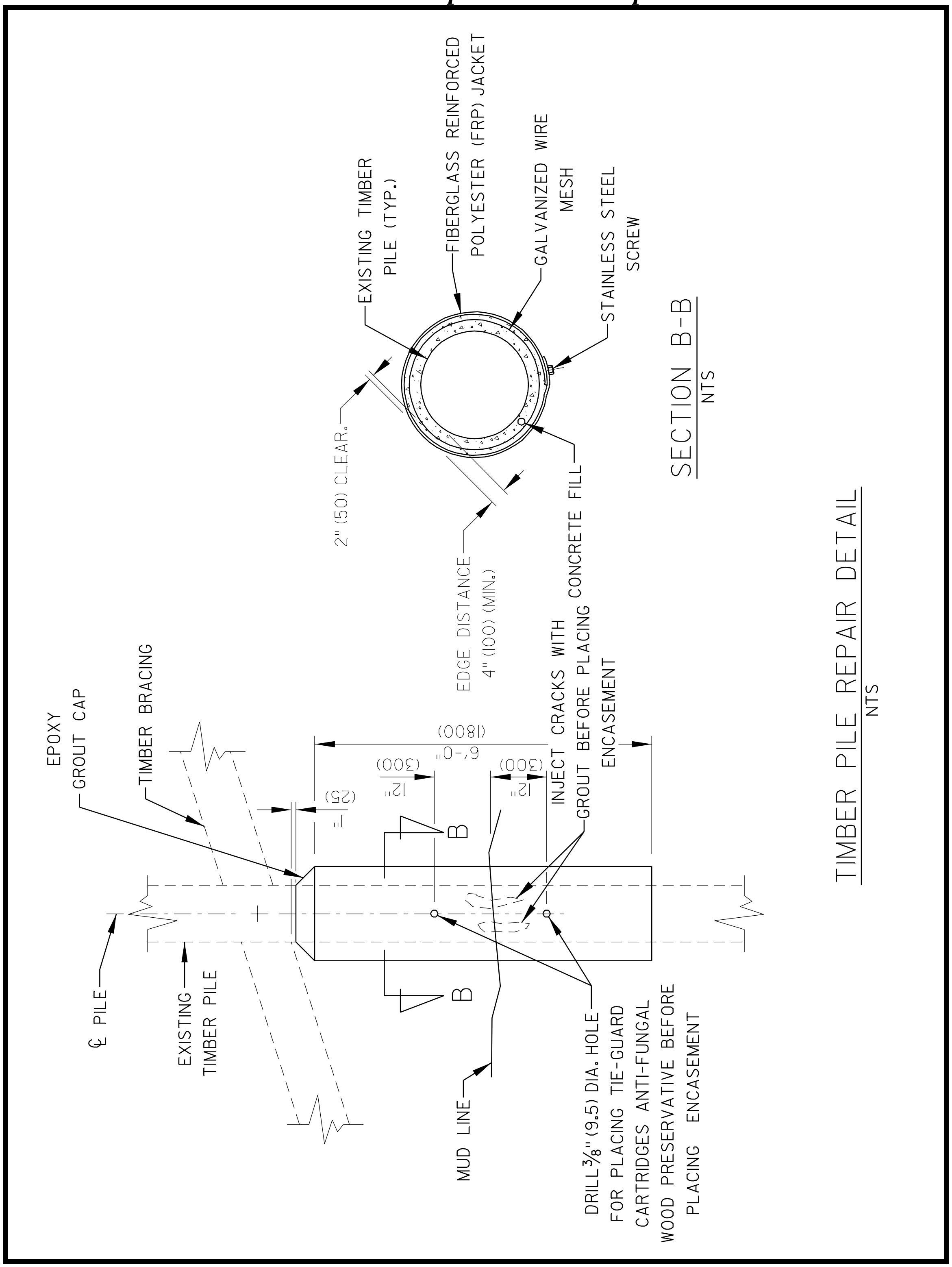


Figure 9-12
Timber Pile Repair Detail Example



9.8.6 SCOUR AND UNDERMINING

Every bridge over a waterway should be evaluated as to its vulnerability to scour. Scour is the result of stream flow that transports streambed material away from the structure. Scour can occur between substructure units or undermine the footings of substructure units. Refer to Section 3.4 for scour evaluation and protection.

Scour has several causes, including:

- poor choice of bridge location,
- meandering river bed,
- inadequate flow area or restrictions imposed by the bridge substructures (substructure size or shape),
- inaccurate estimation of flood level and velocity,
- excessive storm (superflood),
- poor orientation of substructure elements with respect to water flow,
- fine-grained riverbed material that can become waterborne with a small increase in velocity,
- unpredictable increases in the flow caused by debris or the failure of an upstream dam,
- lack of scour prevention measures such as riprap,
- improperly designed riprap where stronger measures, such as sheet piling, should have been used, and
- adding riprap or constructing a cofferdam around a scoured pier may induce undermining of other piers.

Scoured areas can be successfully repaired only if the cause is first identified and understood. The impact of any countermeasures must also be evaluated. Scour countermeasures must be properly designed. Refer to Section 3.4.

Materials such as interlocking concrete elements, cable-tied concrete blocks, revetments, or an articulated grout mattress of quilted cloth and grout have been used. The most commonly used repair methods are:

- riprap,
- bagged concrete, and
- sheeting and filling.

9.8.6.1 Riprap (Stone or Sacked Concrete)

Riprap may be stone, broken concrete, or sacked concrete. Broken concrete must meet the same specifications as stone riprap. The contract documents must specify that concrete from the bridge is to be salvaged for use as riprap, when that use is desired. Sacked concrete is made by filling sacks with a concrete mixture. Hydration occurs after the sacks are placed in the water. Where applicable, dumped or placed riprap of the design size, thickness and layers is considered the most economical method of repairing and preventing scour damage. The riprap is placed to a maximum slope of 1:1. A 2:1 slope is more desirable. Sacked concrete riprap may be used on steeper slopes than stone.

Riprap repair will include the following:

- establish level bearing area, if possible,
- place filter cloth,
- place small bedding stones, and
- place riprap, using smaller stones to key larger stones in place.

Grouting riprap is discouraged.

9.8.6.2 Bagged Concrete

Unfilled bags are placed under a partially undermined foundation and positioned to restore support. Bags are placed in an interlocked configuration to produce a maximum slope of 1:1 when filled. Grout pipes must be placed (prior to filling the bags) to allow grouting of the void spaces behind the bags. Grout pipes are placed between the bags or by coring through the footing and are spaced about 10 feet [3 m] apart. The bags are then filled with tremie concrete or grout by pumping. Vents must be placed to allow water and laitance to be displaced by normal full-strength grout to ensure that non-structural material is not trapped below the footing. When the bags are filled and cured, grout is pumped into the void to fully restore bearing to the footing. Stone riprap is placed to protect the completed grouted bags. The level of the riprap should conform with stream cross section, if possible. Refer to Figure 9-13.

Figure 9-13
Foundation Repairs – Bagged Concrete

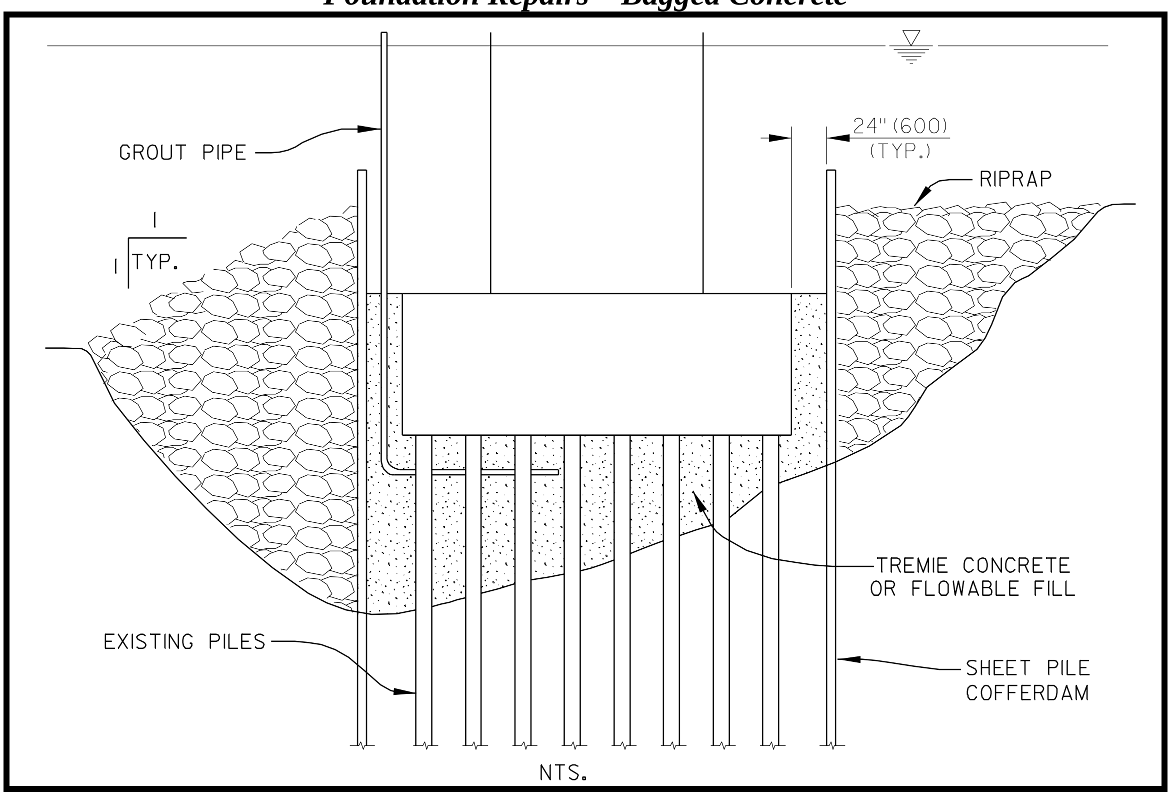
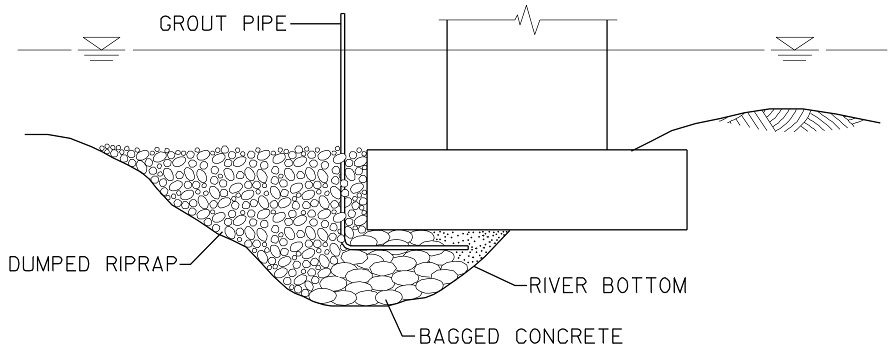


Figure 9-14
Sheeting and Filling



9.8.6.3 Sheeting and Filling

Complete undermining of the foundation requires immediate repair. An undermined foundation is a critical condition. If piles are exposed, lateral and vertical stability of the substructure becomes critical. If the piles are of the friction type, erosion of soil around them lowers their capacity. If they are end-bearing piles, deepening scour could reduce their stability. The unsupported length of piles changes when scour removes the side support and may cause buckling.

To make the repair, steel sheeting is installed around the foundation, often after a section of the deck is removed. Sheeting may be driven from under the deck, if there is sufficient clearance. Short lengths are driven, additional short sections are welded on, and the driving is continued to the appropriate depth.

Tremie concrete or flowable fill is pumped into the sheeted area until it reaches the top of footing. Several pipes may be required. The pipes and the sheeting are then cut off at the desired elevation and riprap is placed around the sheeting. See Figure 9-14. Tremie concrete and flowable fill may bond to existing piles. This will add significant dead load to the piles. A structural analysis must be performed to evaluate the effect of the increase in load.

9.8.7 STABILIZATION AND UNDERPINNING

Substructure units that have experienced foundation displacement (settlement), or where conditions indicate potential displacement, require stabilization and underpinning. Methods of stabilization or underpinning include:

- driven piling and needle beams,

- structural frames, and
- tie backs and deadmen (for abutments).

Both steel and concrete materials are used.

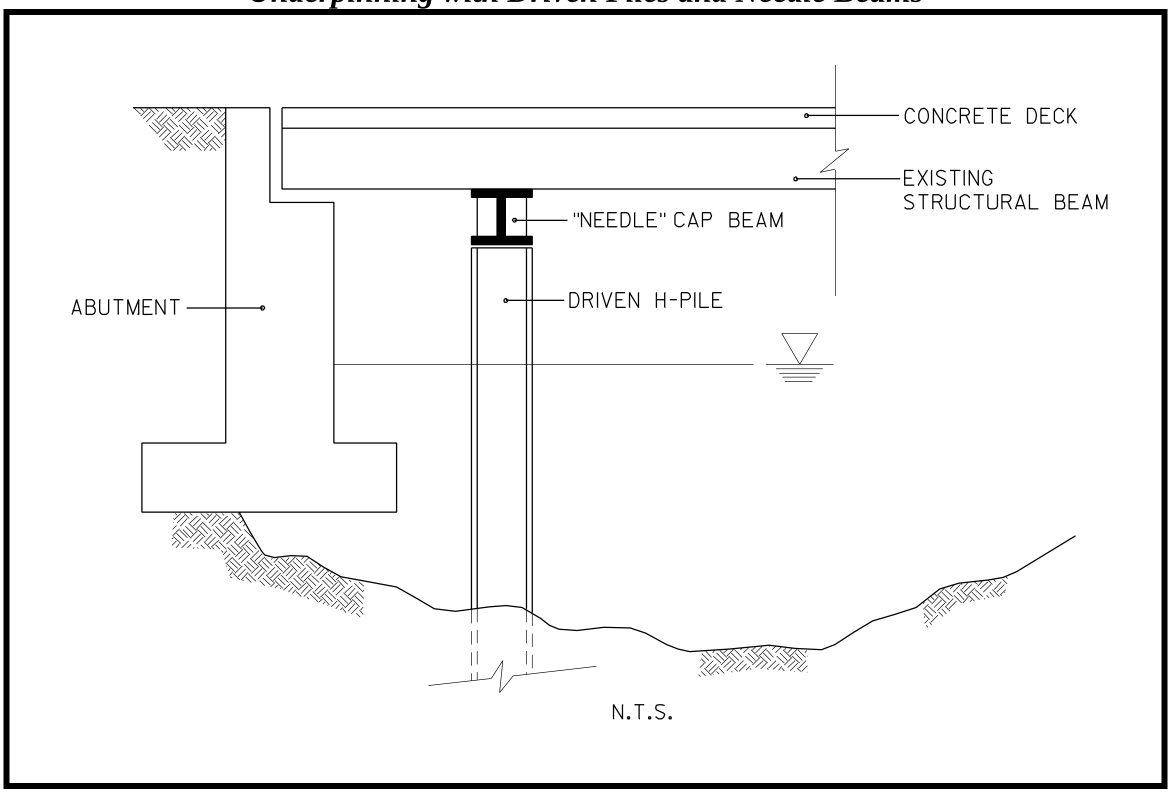
Each of these methods allows supporting the existing superstructure while permanent construction restores the bridge to full traffic.

9.8.7.1 Driven Piling and Needle Beams

Driven piling and needle beams are used primarily to underpin spans that have dropped due to pier settlement. With this method, steel piles are driven, either as a cluster beyond the outside width of the substructure, or as a line of piles driven through the deck of the dropped spans. The piles are cut off and a short "needle beam" is then "threaded" under the span and bears on plates positioned on top of the piles. All connections are made by welding. If necessary, the span can be jacked from the needle beam to restore the original roadway elevation. Anchor bolts are placed to connect the needle beam to the superstructure. Because the piles are placed beyond the width of the pier on both sides, this method may reduce the structure opening and impede stream flow temporarily.

Underpinning with piling and needle beams is normally a temporary repair. If it is to be constructed as a permanent repair, a hydraulic and hydrologic evaluation must be performed. In addition, the piles must be permanently protected or encased. See Figure 9-15.

Figure 9-15
Underpinning with Driven Piles and Needle Beams



9.8.7.2 Structural Frames

When short-span structures settle, temporary support can be achieved using structural frames constructed over or under the spans. These frames can be used to temporarily raise the spans to their original elevation, or to "hold" the structure from further settlement or complete failure.

Support from the top. These structural frames consist of transverse or longitudinal beams that support a single or multiple stringers with hanger rods passing through the deck. The rods support saddles which "cradle" the stringers. Generally, traffic must be removed from the bridge. When longitudinal beams are used, single-lane traffic consisting of lightweight vehicles may be permitted. Refer to Figure 9-16.

Support from the bottom. This type of structural frame supports the span from below and is constructed on cribbing placed directly under the bridge. The cribbing serves to significantly reduce and distribute the load to the soil below. Often this method can be used to avoid a collapse until a permanent support can be constructed. Refer to Figure 9-17.

Jumper bridge. Another type of temporary structural frame consists of multiple beams that are placed longitudinally on top of an existing bridge but span beyond the bridge to create a "jumper" bridge. This method is temporary because it significantly raises the approach roadway to match the jumper bridge and may not be suitable for all locations. Traffic can be temporarily restored quickly with no effect on wetlands or in-stream construction. Refer to Figure 9-18.

9.8.7.3 Tie Backs and Deadmen

The use of tie backs and deadmen is suitable for stabilizing retaining walls and abutments where displacement is caused by overturning. The cause of the displacement must be known or determined before using this method. The method requires excavation behind the structure to place piles, deadmen, and tie rods. Traffic must be restricted until backfilling is completed. Refer to Figure 9-21.

9.8.8 UNDERDRAIN REPAIR

Underdrain systems installed during original construction may not perform for the life of the structure. Leaching water through cracks in abutments or retaining walls indicates that drains are not working properly. When underdrain systems fail, piping water beneath or around the substructure may cause erosion.

Correction of the drainage system failure requires removal of the old system and replacement with new pipe, filter fabric, open-graded stone and backfill.

9.8.9 SLOPE PAVING

The major cause of slope paving failure is poor drainage design resulting in undermining. The designer should determine the cause and provide for correcting the cause in addition to repair of the slope paving.

If the area of slope paving failure is small, in-kind repairs are made. If over 50 percent of the area must be repaired, the slope paving is replaced with riprap.

Figure 9-16
Structural Frames – Top Support

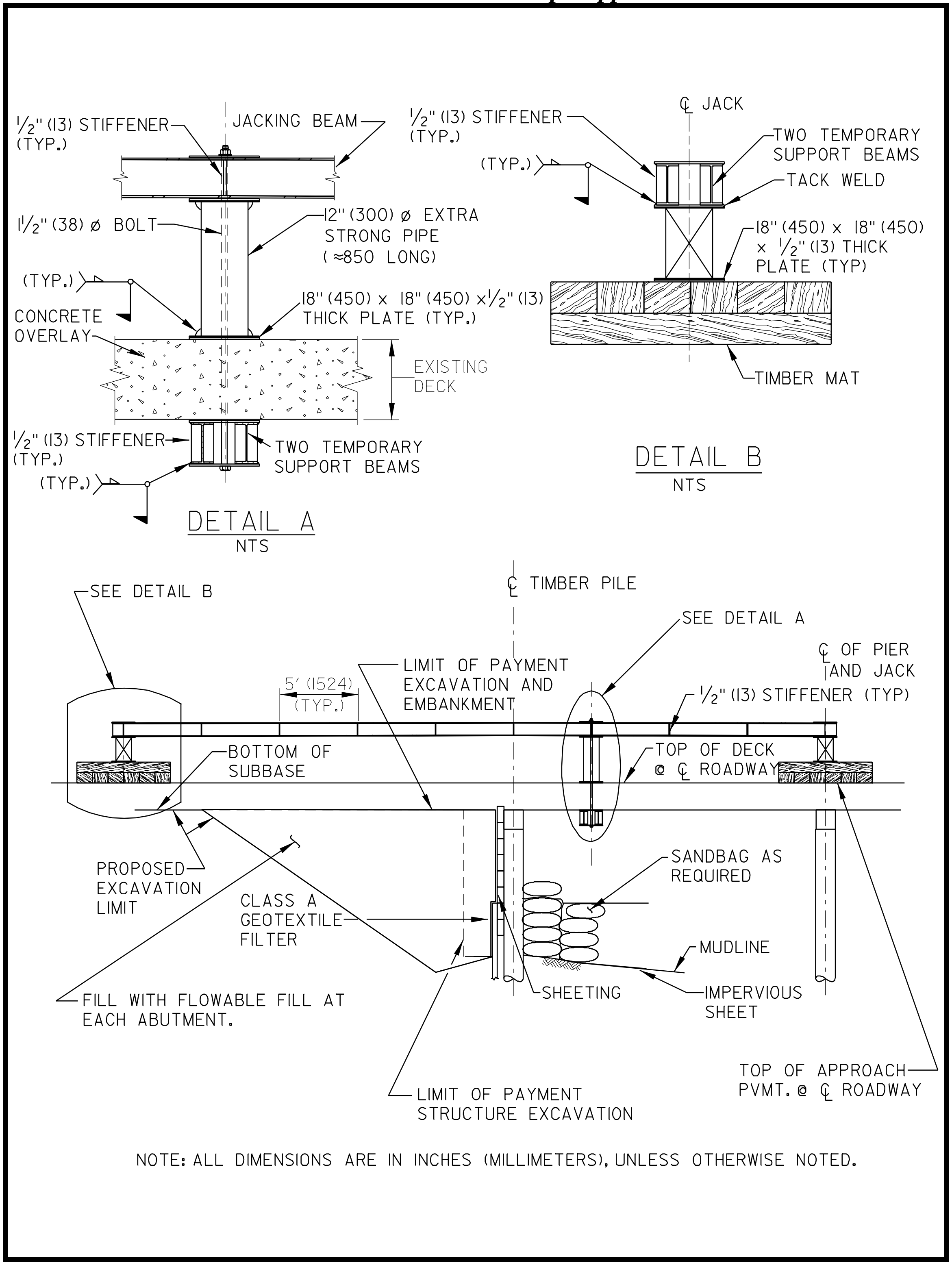


Figure 9-17
Structural Frames – Bottom Support

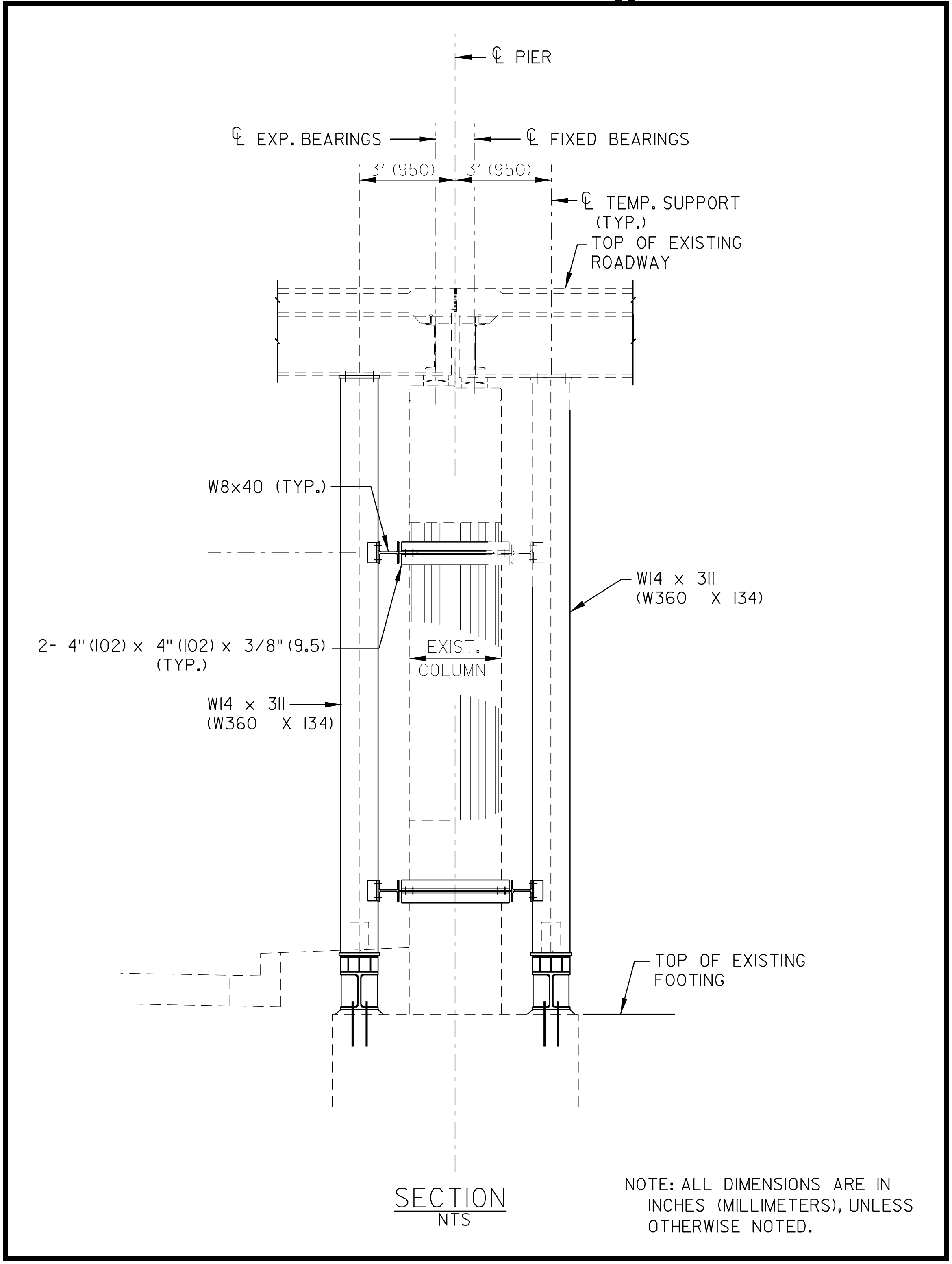


Figure 9-18
Jumper Bridge

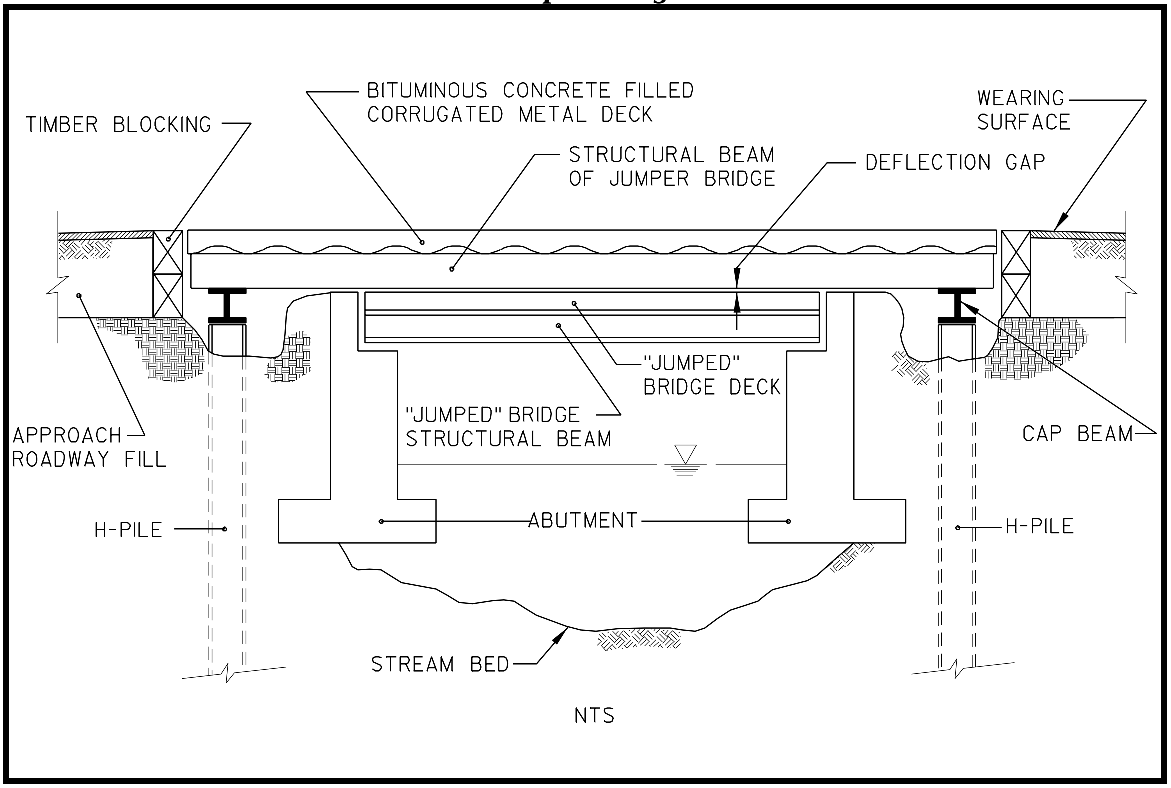
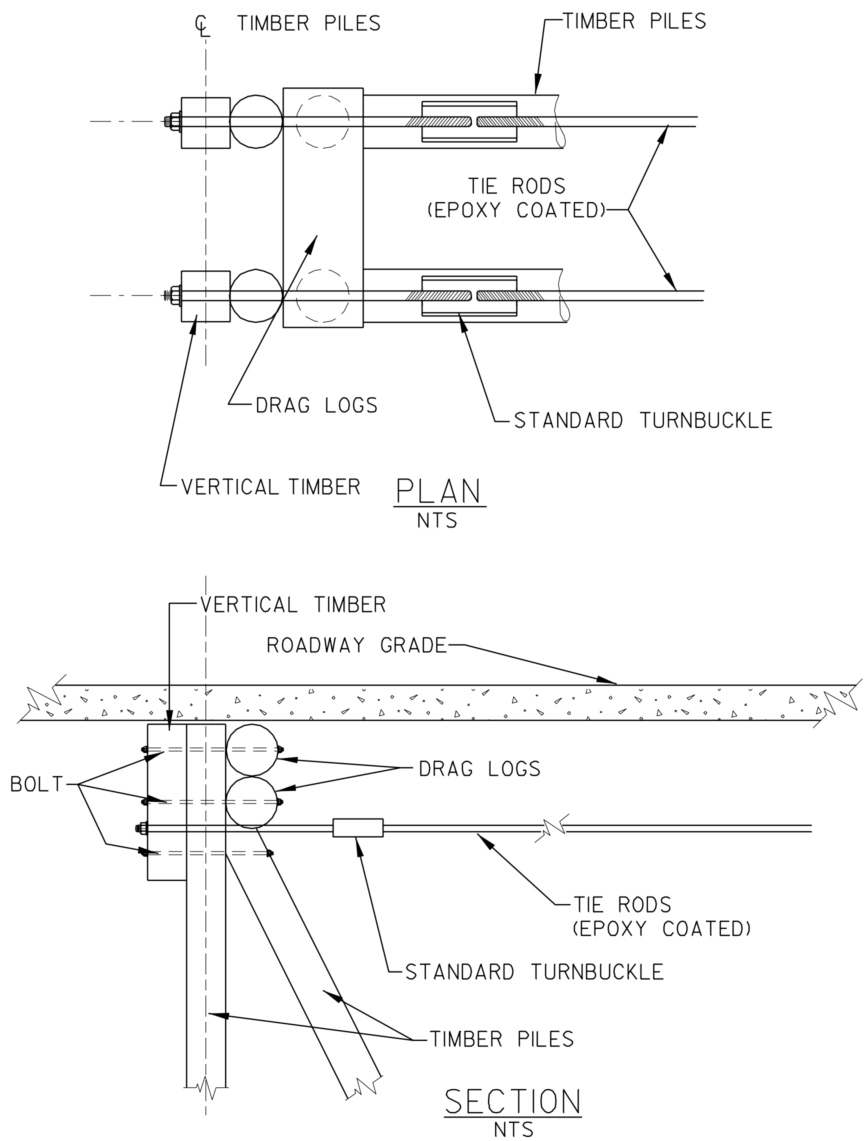


Figure 9-21
Tie Backs and Deadmen



9.9 RETAINING WALLS

Most bridge structures are constructed with wingwalls to retain the roadway and embankments. Also, bridges with long approach embankments may require long walls. Generally, these retaining walls are constructed in one of the following configurations:

- reinforced concrete,
- proprietary such as mechanically stabilized earth (MSE) walls (VSL and Reinforced Earth),
- gravity walls,
- steel sheet piling (tied back and cantilever), and
- gabions.

Because of the differences in the ways they support embankments, each of these types requires special considerations for rehabilitation design.

9.9.1 REINFORCED CONCRETE

The designer should refer to Section 9.8.4 for repair and rehabilitation methods for reinforced concrete retaining walls. Generally, only repairs to the exposed face of the wall can be accomplished.

9.9.2 PROPRIETARY WALLS

Proprietary walls and their application in Delaware are relatively recent. The preferred approach of the Department is to emphasize excellent construction techniques and preventive measures to preclude future problems with MSE walls. The wall types most utilized are VSL and Reinforced Earth.

Reinforced or mechanically stabilized embankments include reinforcing elements such as straps, bars, welded wire mats, polymer grids, geosynthetics, and various anchor systems used to improve the mechanical properties of a large soil mass. Refer to *TRB Circular No. 444, Mechanically Stabilized Earth Walls*, dated May 1995.

Reinforced earth walls are designed and analyzed by considering the entire reinforced mass as a semi-rigid gravity retaining wall with active soil pressure applied behind the wall. The wall must be checked for conventional stability, including overturning, sliding, bearing capacity, and deep stability.

Failures of MSE walls have most recently been limited to failure of the mechanical attachment of the grid, bar, or strap to the facing panel. This failure allows local erosion and displacement of the fill and could cause loss of structural stability over time. Grouted tie-back anchors are used to stabilize and replace MSE wall panels.

The Department does not allow construction of spread-footing-type abutments on MSE embankments. Piles will be used to support the abutment.

In all cases, the manufacturers of MSE walls should be consulted during the development of any remedial repairs.

9.9.3 GRAVITY WALLS

Gravity walls are generally repaired or replaced in kind. Depending on the extent of deterioration, partial reconstruction may be required. With stone masonry gravity walls, repointing or grouting the joint mortar may be required to seal the face.

9.9.4 STEEL SHEET PILING

The primary failure of steel sheet pile walls is the deterioration of the protective coating. Refer to Section 9.7.4.1 for cleaning and painting steel.

9.9.5 GABIONS

Gabions are stacked stone-filled wire baskets interconnected to form gravity-type retaining walls. Settlement is the primary cause of distress. Generally, significant settlement can be tolerated before reconstruction is necessary. Gabion walls are constructed using a 2:1 ratio of height to base width. Corrosion or failure of wire baskets can be repaired by retying the wire. Any displaced stones would be replaced before closing the baskets.

9.10 MAINTENANCE AND PROTECTION OF TRAFFIC

The primary consideration by the Department involving maintaining traffic is to evaluate possible total closure of a structure and detouring all traffic versus maintaining traffic on the bridge. This decision involves the following areas:

- detour availability and compatibility,
- average daily traffic (ADT),
- safety during construction,
- vehicular safety,
- public convenience,
- user costs, and
- construction costs.

Traffic may be detoured to a parallel bridge, if one is available. Detours of pedestrian traffic should also be considered in the project.

Other alternative options to a full detour of all traffic for an extended period include:

- a long-term detour of one direction of traffic while the opposite direction uses the existing structure,
- maintaining a reduced number of lanes on the existing structure, or
- maintaining reduced lane widths on the existing structure.

On a case-by-case basis, a temporary detour bridge may be constructed with the approval of the Bridge Design Engineer.

The designer should refer to the FHWA *Manual on Uniform Traffic Control Devices for Streets and Highways* (*MUTCD*) and the *Delaware Traffic Controls for Street and Highway Construction and Maintenance Operations Manual* for design of traffic control items.

9.11 UTILITIES

Any utilities on an existing bridge must be protected during rehabilitation. Planking or other protection must be provided to prevent damage from dropped items.

The designer should be aware of any utilities near the structure that may affect the contractor’s operation during rehabilitation. For example, high-power electrical lines pose a hazard for crane operation.